UNITED STATES
TENNESSEE VALLEY AUTHORITY

THE NORRIS PROJECT

A Comprehensive Report on the Planning, Design, Construction, and Initial Operations of the Tennessee Valley Authority's First Water Control Project

TECHNICAL REPORT No. 1

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TENNESSEE VALLEY AUTHORITY,
Knoxville, Tenn., March 27, 1939.

Mr. JOHN B. BLANDFORD, Jr., *General Manager,*
Tennessee Valley Authority, Knoxville, Tenn.

DEAR MR. BLANDFORD: The attached report on the planning, design, construction, and initial operations of the Norris project has been prepared [1] by my own staff with contributions from a large number of persons from the various staffs of the Authority.

On account of the widespread interest in the Norris project and its monumental character, I recommend that this report be printed as a public document.

Yours very truly,

T. B. PARKER, *Chief Engineer.*

Approved by Board of Directors, April 17, 1939.

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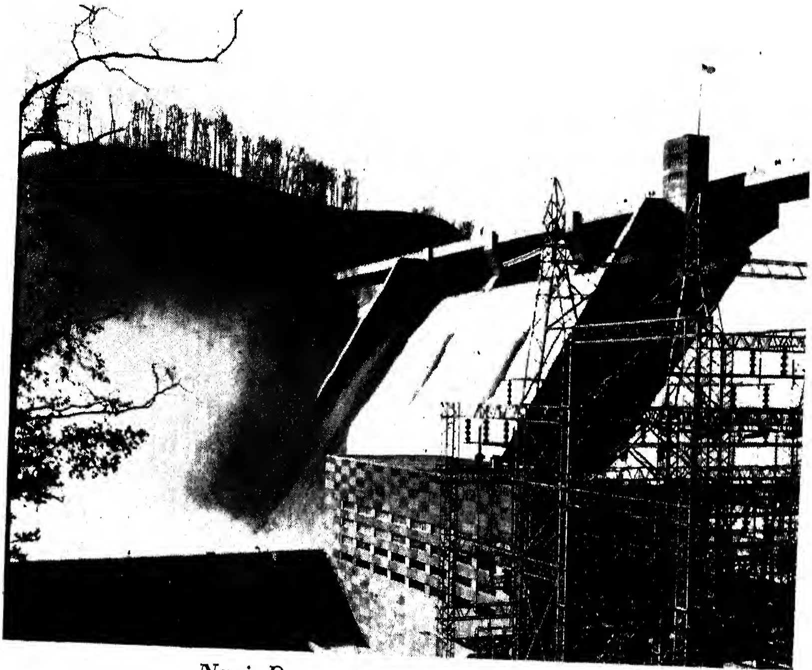

Norris Dam, powerhouse, and switchyard.

THE NORRIS PROJECT

CHAPTER 1

INTRODUCTION

This report is published for the purpose of giving to the engineering profession the important and useful facts about the planning and construction of the Norris Dam and Reservoir on the Clinch River, in eastern Tennessee, by the Tennessee Valley Authority, an agency of the United States Government.

The material presented herein has been selected and condensed from the large mass of data, many times greater in volume, contained in the Authority's files. To make this report of greatest use to those engaged on other similar projects, relatively little space is given to such parts of the work as followed well-established engineering practice; while, on the other hand, novel or unprecedented features are described and explained in much more detail. It is hoped that many parts of this exposition will be found useful and interesting also to the general public.

LOCATION AND CHARACTER OF THE TENNESSEE VALLEY

The headwaters of the Tennessee River are in the Great Smoky and the Blue Ridge Mountains—the highest ranges east of the Rockies—which are located in eastern Tennessee, in western Virginia and North Carolina, and in northern Georgia. The main river begins just above Knoxville, Tenn., at the confluence of the Holston and French Broad Rivers. It flows in a southwesterly direction through Tennessee, crosses northern Alabama, forms a small portion of the northeastern boundary of Mississippi, and then flows north through western Tennessee and western Kentucky to empty into the Ohio River at Paducah, Ky., a distance of about 650 miles. The Tennessee River drains an area of 40,910 square miles—about equal to four-fifths of the area of England. This is a region of forests, pasture lands, orchards, and small farms. The mountain region, which includes the Great Smoky Mountains National Park and a number of national forests, is in striking contrast to the relatively flat lands of northern Alabama, occupied by large cotton plantations, and to the rolling land of western Kentucky. The Tennessee Valley area has a population of approximately 2½ million people, of whom only about a quarter live in cities; about half are actually farm dwellers.

The Tennessee River drainage basin has a variety of natural resources. About half of its area is covered by forests from which most of the merchantable timber has been cut and the region has a large supply of coal, iron, and various other minerals. The fall of the streams from an elevation of more than 3,000 feet in the upper valleys to about 300 feet at the outlet of the Tennessee River into the Ohio, combined with the large annual rainfall, which ranges from 40 to 80 inches, provides great water power possibilities.

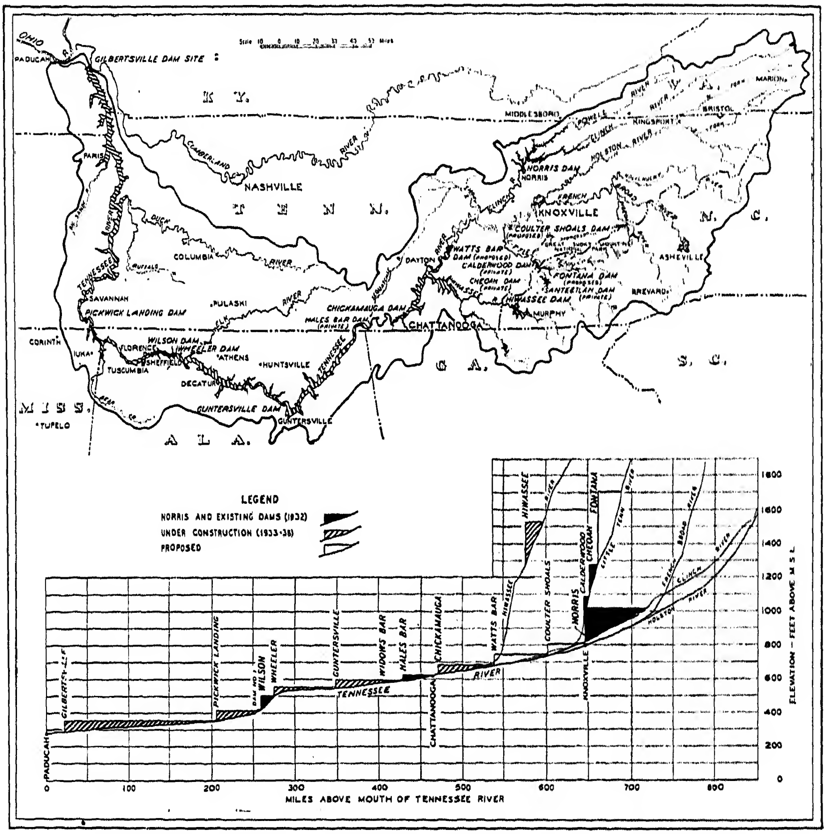

FIGURE 1.—*Map and profile of the Tennessee River system.*

The general program of the unified development of the Tennessee River system is outlined in the Authority's report [1] to Congress dated March 31, 1936. This program will provide a navigation channel of liberal capacity from Knoxville to the mouth of the river, a distance of about 650 miles. It will insure an adequate water supply for the operation of locks and will contribute substantially to flood control. It will also provide many incidental benefits of importance, including a large supply of electric power.

In addition to the main river dams, the plan already provides for large tributary dams. These dams will store floodwater and thus provide substantial control for the regulation of high water fluctuations at dams and river terminals below and for an increased low water flow. Facilities are being planned for producing electric power at each of these tributary dams. Norris Dam, the first to be constructed, plays an important part in this system because of its large storage capacity and the comparatively large flow contributed by the Clinch and Powell Rivers to flood peaks on the Tennessee River.

HISTORY OF THE TENNESSEE RIVER DEVELOPMENT

The history of the Norris project is inextricably connected with that of the Muscle Shoals developments in Alabama. The importance of navigation on the Tennessee River has been recognized for more than a century. In 1824 John C. Calhoun, then Secretary of War, recommended its improvement to President Monroe as part of a broad program of waterways development.[2]

In 1828 the Congress authorized surveys and estimates for a canal and locks around Muscle Shoals, and in 1831 construction was begun by the State of Alabama with funds realized from the sale of 400,000 acres of land donated by the Federal Government for that purpose. A lateral canal with 17 small locks, constructed throughout the length of Muscle Shoals, was opened to navigation in 1834. In 1871 the Congress approved the construction of a larger Muscle Shoals Canal, and in 1890 provided for further improvement in the Colbert Shoals section by a lock and a canal with width of 112 feet and depth of 7 feet.

Navigation was the only problem the Tennessee River presented for solution by the Federal Government between 1826 and 1899. In 1899, however, the Congress gave approval to the Muscle Shoals Power Co. for constructing a canal and power station at Muscle Shoals. The company never took advantage of that authorization although the time for beginning the project was successively extended by three later acts.

In 1903, the same year in which this last extension of time was given, the Congress passed a bill granting a franchise to another group. This bill received an unexpected veto, the first of its kind, from President Theodore Roosevelt. In vetoing this bill he expressed his belief that the Government should develop the water power as a part of its navigation improvement and that no private license should be granted except on the payment to the public for the use of any water-power rights conferred. His veto message stated, in part, that:

> The recent development of the application of water power to the production of electricity available for use at considerable distances has revealed an element of substantial value in streams which the Government is or is liable to be called upon to improve for purposes of navigation, and this value, in my judgment, should be properly utilized to defray the cost of the improvement.

Power, only one of the many phases of water control which enter into the complete picture, bears a special relation to the others.

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[2] Eighth Annual Message of President James Monroe.

Power rights, being valuable, have been coveted by private interests, and their utilization by the Government has been bitterly attacked.

President Theodore Roosevelt's views on this subject were shared by his successor, President Taft, in his veto of a bill for the private construction of a dam on the Coosa River in Alabama in 1912. The wisdom and the right of the Government to pursue this course was endorsed by other leading public men of the day. Elihu Root, in a Senate debate in 1913, urged that the Government avail itself—

* * * of this new discovery by which a stream can be made to improve itself, by which a stream can be made to pay the expense of fitting itself for navigation, * * *.

These views represented the logical application of principles enunciated in earlier opinions[3] of the Supreme Court. The right of the Federal Government to use and dispose of the power created at Government projects was upheld as an inevitable attribute of its full and complete navigation power. In upholding the disposition of the water power by the Government, the Supreme Court further pointed out that the Government might thus reimburse itself for the expense of the improvement, and this decision has been reaffirmed several times.

Moreover, the Court has refused to recognize the existence of vested rights which would make the expense of Government improvements prohibitive. In the Chandler-Dunbar condemnation case in 1913 it established principles to prevent such a barrier to public development. The Court denied the contention that there could be any private ownership of the water or the power in a navigable stream as against the Federal Government. This court decision may be considered as having a direct bearing upon the Norris project. The Clinch River, both above and below the Norris Dam site, has been considered a navigable stream for more than 100 years. Provisions have been made for the possible future building of a chute through the dam for passing coal—the stream's principal prospective tonnage.

President Theodore Roosevelt's Conservation Commission recommended in 1909 a comprehensive and unified development of our rivers:

Broad plans should be adopted providing for a system of waterway improvement extending to all uses of the waters and benefits to be derived from their control, including the clarification of the water and abatement of floods for the benefit of navigation; the extension of irrigation; the development and application of power; the prevention of soil wash; the purification of streams for water supply; and the drainage and utilization of the waters of swamp and overflow lands.

The National Waterways Commission, created by the Congress in 1912, forecast that the Federal Government must inevitably undertake the unified development of river systems by means of multipurpose projects.

In 1907 the Secretary of War appointed a special board of engineers to examine the Muscle Shoals section "with a view to permitting the improvement of the above-described stretch of said river by private or corporate agency in conjunction with the development

[3] *Kaukauna Water Co.* v. *Green Bay and Miss. Canal Co.*, 142 U. S. 254, 273 et seq.; *Green Bay and Miss. Canal Co.* v. *Patten Paper Co.*, 172 U. S. 58, 81 (1898), rehearing denied, 173 U. S. 179 (1899).

of water power by means of not more than three locks and dams". This survey was the first to be undertaken by the Government in contemplation of a combination water power and navigation development in the Muscle Shoals section. It was made at that time because a bill had been introduced to permit the Muscle Shoals Hydro-electric Power Co. to erect three dams at Muscle Shoals for the generation of hydroelectric power and the improvement of navigation, with the expenditure to be divided between the company and the Federal Government.

This board of engineers submitted a report[4] in 1908 which disapproved the plans proposed by the power company on the ground that they were not sufficiently satisfactory from the viewpoint of navigation or in consideration of the investment which the company proposed to make. Navigation was still the decisive factor insofar as the interest of the Federal Government was concerned, but water power as an issue had come to stay.

The offer was re-examined and again rejected[5] by the special board in 1909, but in 1914 the Chief of Engineers recommended[6] that a later bid of the power company for making the proposed improvements should be accepted with certain reservations. This offer was judged to be more generous than the earlier proposal, and in addition the Board of Engineers took into consideration the public demand that the water power resources of the country should be more fully utilized.

A congressional committee requested additional information before making a final decision on the proposal and again the Board of Engineers conducted a re-examination and resurvey and in June 1916 concurred[7] in previous recommendations that the Government should enter into a plan of development with the power company whereby "this serious obstruction to navigation on the Tennessee River will be overcome and at the same time one of the great natural resources of the country will be conserved".

The issues raised by this proposal were never settled, for with the report came a recommendation that all negotiations should be suspended as "the board invites attention to the provisions contained in section 124 of the Act to Increase the Efficiency of the Military Establishment of the United States, approved June 3, 1916, respecting the establishment of nitrate plants to supply Government needs in the manufacture of explosives for military uses". Under this act the President was authorized to select a site and choose a method for the production of nitrates essential for munitions of war and useful in the manufacture of fertilizer and other products in time of peace. President Wilson chose the Muscle Shoals site in September 1917.

The War Department was assigned the task of carrying out the construction program. The entire plans for power development included the construction of three dams with locks for navigation and power stations for the generation of electricity at two of them. Before Wilson Dam was completed, the World War had ended and work thereafter was continued intermittently, being interrupted twice by

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[4] H. Doc. No. 781, 60th Cong., 1st sess.
[5] H. Doc. No. 14, Committee on Rivers and Harbors, 60th Cong., 2d sess.
[6] H. Doc. No. 20, Committee on Rivers and Harbors, 63d Cong., 2d sess.
[7] H. Doc. No. 1262, 64th Cong., 1st sess.

the failure of the Congress to appropriate funds. It was finally completed during the latter part of 1925. The construction of Lock and Dam No. 1, a small navigation project 2.55 miles downstream from Wilson, was authorized by the Congress in 1925 and was completed

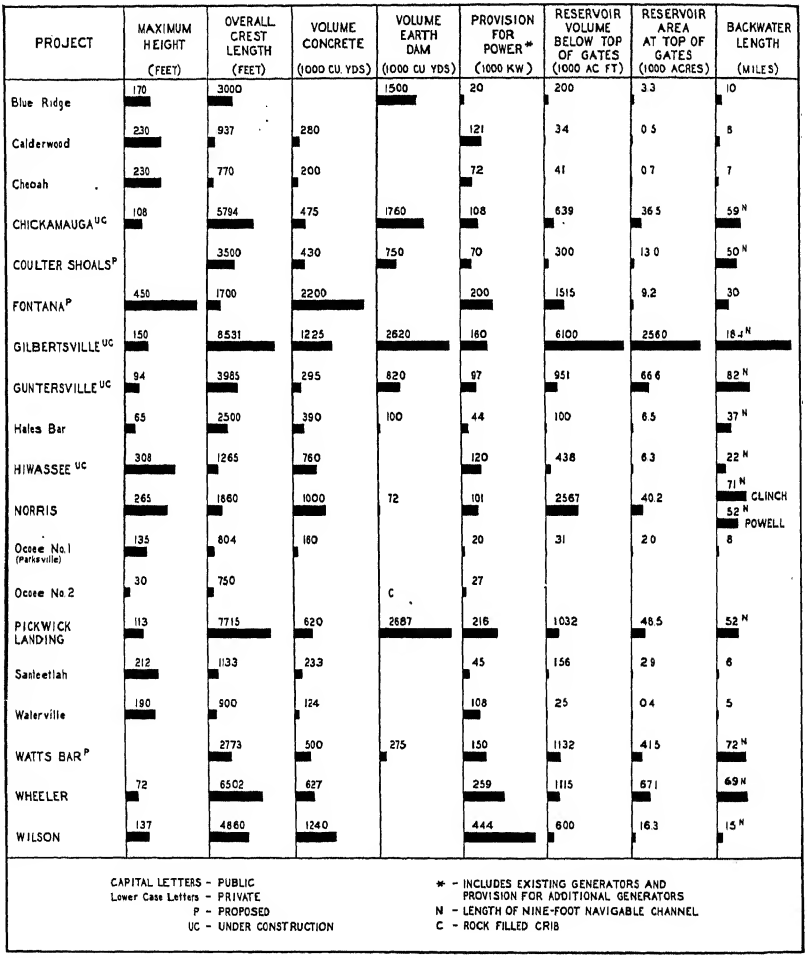

Figure 2.—*Water-control projects in the Tennessee Valley—A comparison of principal features.*

shortly afterward. Construction work on the lock for Dam No. 3 (Wheeler) was not started until January 1933.

Wilson Dam, as a power project, was seriously handicapped by the exceedingly variable flow of the river, whose maximum recorded

flow of 444,000 cubic feet per second was approximately 100 times the recorded minimum. Having only limited storage capacity, it offered very little assistance to navigation except within its own locks and reservoir, and during low-water season even the smaller commercial boats experienced considerable difficulty in reaching the Wilson locks from points downstream.

It immediately became evident that the Tennessee River should be more effectively regulated to conserve the water resources of the basin, to improve navigation both above and below Wilson, and to provide some appreciable measure of flood control. Upstream storage projects appeared to be necessary for any complete solution of the problem.

EVOLUTION OF THE NORRIS PROJECT

The rise of the idea of a coordinated plan for the development of the Tennessee River projects inevitably tied the Cove Creek (Norris) project with the Muscle Shoals properties and any other projects developed on the main river.

The Clinch Basin seemed to offer excellent opportunities for construction of large storage projects. As early as 1911 the present site of Norris Dam was investigated [8] by power interests. The first study [9] of the river and its tributaries with the idea of the coordinated development of these possibilities was presented in 1918 in a paper [10] of the Tennessee State Geological Survey. A similar study was included in a preliminary report [11] submitted by the United States Army Engineers in 1922. These studies recommended a number of dam sites, among them one on the Clinch River at approximately the present location of Norris Dam and known at that time as the Cove Creek site.

At the time of the creation of the Tennessee Valley Authority, several reservoirs existed in the Tennessee Valley above Wilson Dam. The Tennessee Electric Power Co. operated Hales Bar Dam and powerhouse which had been built between 1905 and 1913 on the Tennessee River just below Chattanooga, and by 1930 had completed a three-dam development on the Ocoee-Toccoa River, an upstream tributary. The Aluminum Co. of America had obtained development rights on the Little Tennessee River system as early as 1910, and by 1930 had constructed three dams and had planned ultimate construction of two or three additional ones. In 1930 the Carolina Power Co. had completed the Waterville project on the Big Pigeon River, a tributary of the French Broad River. Numerous other smaller water-power plants had also been completed by private interests. Further preliminary power studies had been conducted by private power companies on the possible hydroelectric development of the French Broad, Holston, and Clinch Rivers.

Several applications were filed by private corporations with the Federal Power Commission for permits to construct dams on the Clinch River. The annual reports of the Commission contain refer-

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[8] Jones, Barton M., Design of Norris Dam and Powerhouse, Civil Engineering, April 1935.

[9] Hearings before House Committee on Military Affairs, January 25–February 1, 1937, p. 447.

[10] Switzer, J. A., Bulletin No. 20. Tennessee State Geological Survey.

[11] H. Doc. No. 319, 67th Cong., 2d sess.

ences to the following applications pertaining to Clinch River developments:[12]

	Application number	Date filed	Withdrawn or rejected
Tennessee Hydro-electric Co	381	Jan. 24, 1923	Aug. 13, 1930
Knoxville Power & Light Co	410	Apr. 21, 1923	July 27, 1923
Tennessee Electric Power Co	427	July 12, 1923	Dec. 8, 1925
Knoxville Power & Light Co	430	July 29, 1923	Do.
East Tennessee Development Co	638	Aug. 26, 1925	Aug. 26, 1930
Union Carbide Co	686	Jan. 14, 1926	Apr. 3, 1934

In proposals for the purchase of the Muscle Shoals properties from the United States, provisions were made by the Union Carbide Co. and others for the building of a dam at the Cove Creek site to increase the minimum flow in the main river and thus increase the primary power at Wilson and other downstream power plants. The eastern Tennessee power interests, which favored development of the Tennessee River system by local power companies, offered many objections[13] to the Union Carbide Co. proposals. Their principal objection was that this company was interested in the Cove Creek project only because of its effect on the Wilson plant and a proposed plant above Wilson whereas they were interested in developing the Clinch and other tributary rivers in a more local and independent manner.

As early as 1922, the outstanding importance of the Cove Creek Dam as a flood-control measure was emphasized, particularly by Senator Norris, who stated that—

> If the improvement contemplated by this bill were made, we would have the maximum amount of electric energy developed on the Tennessee River at the minimum cost, and it would follow that by the storage of the floodwaters we would automatically assist to a great extent in lessening the danger of overflow on the Ohio or the Mississippi. It is one of the incidents that goes with the systematic improvement of any navigable stream. All these beneficial objects interlock and work together for the development of power, the improvement of navigation, and the avoidance of damage by overflow. In this way we are getting every possible benefit that can come through improvement.

Pursuant to the River and Harbor Acts of 1922 and 1925, the Army Engineers made an extensive study of the navigation, flood control, and hydroelectric possibilities of the Tennessee River system in 1926. In the report[14] submitted, specific mention of the Cove Creek Reservoir was made and an estimate of its probable cost was given. This preliminary report was followed in 1928 by the more detailed "Report on Cove Creek Dam site and recommendation for design of dam, powerhouse, barge lift, and spillway".[15]

During the Sixty-ninth Congress, Second Session, 1926–27, the proposed Cove Creek Dam figured in four measures. The first, Senate Joint Resolution 167, presented by Senator McNary, provided for completion of the survey of the dam and reservoir on the Clinch

[12] See Annual Reports, Federal Power Commission.
[13] Committee on Agriculture and Forestry, 69th Cong., 1st sess., Senate. Hearings * * * to suspend jurisdiction of the Federal Power Commission to issue license on Tennessee River watershed. January 11, 18, 19, 1926.
[14] H. Doc. No. 463, 69th Cong., 1st sess.
[15] H. Doc. No. 185, 70th Cong., 1st sess.

River. It was referred to the Committee on Commerce on February 19, 1927, and died on the calendar. There was also notice of a proposed motion in the Senate to suspend rules and offer an amendment to the second deficiency appropriation bill (H. R. 17291) providing an appropriation for investigations with respect to the proposed Cove Creek Dam. The two measures which stimulated more discussion and research than any others during this Congress were H. R. 16396 and H. R. 16614. The first of the two was a bill providing for, among other things, the completion and maintenance of the Muscle Shoals program; the other authorized the Secretary of War to lease Muscle Shoals properties, as well as points on the Clinch River, to Air Nitrates Corporation and American Cynamid Co. Hearings were held on these two bills covering four volumes of 2,871 pages, of which more than 88 pages, involving in excess of 46 separately indexed references, were concerned with Cove Creek Dam. In reporting on the two bills, it was proposed that the Secretary of War be requested to allot funds for a preliminary survey of the Cove Creek project. This session of Congress adjourned without enactment of the measures.

The first bill to authorize actual construction of the Cove Creek Dam by name was the Muscle Shoals bill of the Seventieth Congress, First Session. It was Senate Joint Resolution 46, presented December 5, 1927, and was a slight modification of the one introduced in the previous session. The bill appropriated two million dollars to begin dam construction during the fiscal year 1929 and stipulated that it be built by Army Engineers and controlled by the Muscle Shoals Corporation. After a series of debates, conferences, and reports,[16] it was finally pocket-vetoed[17] by President Coolidge.

The flood-control and navigation aspects of the Cove Creek project were coming into increased importance. With reference to Cove Creek Dam Senate Report 19 on Senate Joint Resolution 49, Seventy-first Congress, First Session, pages 10–12, states:

> * * * under no circumstances should the Government ever permit a private corporation to build this dam (Norris) for the purpose of generating power. If it is used solely as a power-development proposition, its value in flood control and navigation will be completely destroyed. If a private corporation should build this dam, it would permit the reservoir to fill with water and remain full, so that it could get the entire amount of fall from the regular flow of the stream. When the floodwaters come, they would go over the dam and do their work of destruction just the same as though the dam were not there. Instead of regulating the Tennessee River for navigation purposes, such a dam, operated for power exclusively, would not increase the flow of the Tennessee in low water and not decrease it in high water. On the other hand, if it were operated as a flood-control and navigation proposition, the reservoir would be emptied once every year. Its waters would be used to increase the flow of the Tennessee when more water was desired, and the waters would be held back when the Tennessee was high and when additional water would do damage rather than bring benefits.

On March 15, 1930, the Chief of Engineers submitted a report[18] on the Tennessee River and tributaries, covering navigation, flood control, and power development. Recommendations and cost esti-

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[16] Congressional Record, Vol. 69, pp. 4635–4636, 9341–9344, 9696, 9704, 9842, 9957. See also S. Rept. No. 228, 70th Cong., 1st sess.

[17] The effectiveness of a pocket veto at the close of the first session of a Congress was upheld in *Okanogan Indians* v. *United States*, 279 U. S. 655 (1929).

[18] H. Doc. No. 328, 71st Cong., 2d sess.

mates for the Cove Creek project, based on investigations that included a geologic examination of the dam site and preliminary surveys of land values in the reservoir area, were included in detail.

In the Seventy-first Congress, Second Session, Senator Norris presented Senate Joint Resolution 49, the same bill vetoed by President Coolidge, with the modification that the State of Tennessee should receive 5 percent of the revenue from sale of power at Cove Creek Dam when completed. It was adopted by the Senate in April 1930 but rejected by the House which substituted H. R. 11585 providing for lease to private companies and construction of the dam by "holding company or otherwise". After a series of hearings and reports,[19] the two bills went to conference and the Congress adjourned on July 3, 1930, with no report having been made to either house.

Another compromise bill providing for the Cove Creek project was agreed upon after much deliberation by the Seventy-first Congress, Third Session. In conference it was finally decided to let the provisions of the Norris bill (S. J. Res. 49) remain with respect to power; but with respect to fertilizer it was modified so as to give the President authority to lease the nitrate facilities for production of nitrates and fertilizers. The report was adopted but the measure vetoed [20] by President Hoover on March 3, 1931. A favorable vote on the vetoed measure was not sufficiently large to make it law.

President Hoover presented to the Seventy-second Congress, First Session, the recommendation of the Muscle Shoals Commission, a document which included in its provisions the construction of the Cove Creek Dam by the Government for purposes of "navigation, flood control, and incidental power development".[21] Several Muscle Shoals bills were introduced in the House during the same session. The most important bill was H. R. 11051, the Hill bill, which provided for the construction of the Cove Creek Dam if the added power to be gained thereby should be found necessary for the making of fertilizers. The bill passed the House on May 5, 1932, and was sent to the Senate Committee on Agriculture and Forestry where it died with the Seventy-second Congress. The House debates during this session, featuring prominently the Cove Creek Dam issue, the Norris bill in the Senate, and the Bankhead bill in the Senate, are all of interest to those who wish the complete story of the Cove Creek Dam legislation. Senator Norris' bill was identical with the Hill leasing bill (H. R. 11051) and was reported favorably in the Senate but died in conference. Senator Bankhead's bill was a substitute for the Norris measure involving the lease of Muscle Shoals, but it did not come up for action.

On April 10, 1933, shortly after his inauguration, President Franklin D. Roosevelt delivered the following message to Congress:

> The continued idleness of a great national investment in the Tennessee Valley leads me to ask the Congress for legislation necessary to enlist this project in the service of the people.

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[19] Congressional Record, Vol. 72, pp. 6399, 6494, 9767, 71st Cong., 2d sess. See also H. Rept. No. 1430, 71st Cong., 2d sess.

[20] S. Doc. 321, 71st Cong., 3d sess.

[21] It is interesting to note that this commission appointed by President Hoover used the phrase "incidental power". The Authority's use of this phrase to describe its power operations has been criticized by the utilities who have contended that power operation is the chief purpose of the Authority and not an "incidental product".

It is clear that the Muscle Shoals development is but a small part of the potential public usefulness of the entire Tennessee River. Such use, if envisioned in its entirety, transcends mere power development: it enters the wide fields of flood control, soil erosion, afforestation, elimination from agricultural use of marginal lands, and distribution and diversification of industry. In short, this power development of war days leads logically to national planning for a complete river watershed involving many States and the future lives and welfare of millions. It touches and gives life to all forms of human concerns.

I, therefore, suggest to the Congress legislation to create a Tennessee Valley Authority—a corporation clothed with the power of Government but possessed of the flexibility and initiative of a private enterprise. It should be charged with the broadest duty of planning for the proper use, conservation, and development of the natural resources of the Tennessee River drainage basin and its adjoining territory for the general social and economic welfare of the Nation. This authority should also be clothed with the necessary power to carry these plans into effect. Its duty should be the rehabilitation of the Muscle Shoals development and the coordination of it with the wider plan.

Many hard lessons have taught us the human waste that results from lack of planning. Here and there a few wise cities and counties have looked ahead and planned. But our Nation has "just grown." It is time to extend planning to a wider field, in this instance comprehending in one great project many States directly concerned with the basin of one of our greatest rivers.

This in a true sense is a return to the spirit and vision of the pioneer. If we are successful here we can march on, step by step, in a like development of other great natural territorial units within our borders.

On April 11, 1933, Senator Norris proposed a bill (S. 1272)

* * * to improve the navigability and to provide for the flood control of the Tennessee River, and to provide for reforestation * * *

This bill and the numerous amendments that were offered were referred to the Committee on Agriculture and Forestry. Also, on April 11, Congressman McSwain offered a bill (H. R. 4859) in the House

* * * to aid interstate commerce by navigation; to provide flood control; to promote the general welfare by creating the Tennessee Valley Authority; * * *

which was referred to the Committee on Military Affairs. On April 24, 1933, after lengthy discussion and debate, Congressman Rankin introduced the Norris bill (S. 1272) in the House as H. R. 5081 which was also referred to the Committee on Military Affairs.

On May 9, after much debate, the House of Representatives appointed three members to meet with conferees from the Senate to adjust the differences between the two houses. On May 15 this joint committee recommended approval of H. R. 5081 including the Senate amendment with slight revisions. This conference report was approved by both houses on May 17, 1933. On May 18, 1933, the bill was signed by the Speaker of the House, the Vice President, and the President.

In general, the act [22] provides for:

* * * maintaining and operating * * * properties * * * in the interest of the national defense and for agricultural and industrial development, and to improve navigation in the Tennessee River and to control the destructive flood waters in the Tennessee River and the Mississippi River Basins * * * [23]

Among the specific provisions the Tennessee Valley Authority was authorized:

* * * to construct, either directly or by contract to the lowest responsible bidder, after due advertisement, a dam in and across Clinch River in the State of Tennessee, which has by long custom become known and designated as the Cove

[22] A copy of the Tennessee Valley Authority Act appears in appendix J.
[23] See sec. 1.

Creek Dam, together with a transmission line from Muscle Shoals, according to the latest and most approved designs, including powerhouse and hydroelectric installations and equipment for the generation of power, in order that the waters of the said Clinch River may be impounded and stored above said dam for the purpose of increasing and regulating the flow of the Clinch River and the Tennessee River below, so that the maximum amount of primary power may be developed at Dam No. 2 and at any and all other dams below the said Cove Creek Dam: * * *[24]

The name of the Cove Creek project was changed to the Norris project by the TVA Board of Directors on July 30, 1933, in honor of Senator George W. Norris of Nebraska.

Immediately after the Tennessee Valley Authority was organized, preparations were begun for the construction of the Norris Dam. A contract was completed with the United States Bureau of Reclamation on September 15, 1933, under which the Bureau agreed to prepare the detailed designs and contract drawings for the dam. Actual construction was begun October 1, 1933. On March 4, 1936, the outlet conduit gates in the dam were closed and storage of water in the reservoir began. On June 19, 1936, the water level in the reservoir had risen to elevation 1,000 above sea level, or 20 feet below the level of the spillway crest, and the first water was released to increase the low-water flow in the Tennessee River for the benefit of navigation. The first generator at Norris Dam was put in service on July 28, 1936.

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[24] See sec. 17.
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Chapter 2

PROJECT INVESTIGATIONS

PRELIMINARY STUDIES AND DESIGNS

GENERAL CONSIDERATIONS

Studies of available dam sites on the Clinch River for the development of power had been made as early as 1911. Various dam sites were recommended including a dam 165 feet high at the present site of Norris Dam supplemented by two larger dams in the upper reaches of the reservoir on the Clinch and Powell Rivers, respectively. In 1913 the United States Army Engineers undertook a survey of the Tennessee River and its tributaries to investigate the flood control, navigation, and power possibilities.

Later navigation, flood control, and power studies of the Clinch and Powell Rivers made by the United States Army Engineers revealed numerous possible dam sites, and a dam at the Cove Creek (Norris) site, among others, was recommended in its last report.[1]

Determination of location.

Following the reports of the Army Engineers, the Congress authorized, in the Tennessee Valley Authority Act, the construction of a dam at the Cove Creek site. Section 17 of the act, referring to the construction, reads in part as follows:

> * * * to construct, either directly or by contract to the lowest responsible bidder, after due advertisement, a dam in and across Clinch River in the State of Tennessee, which has by long custom become known and designated as the Cove Creek Dam. * * *

Further power for a Governmental bureau, or the board, and directions in regard to the use were contained in the following section:

> SEC. 18. In order to enable and empower the Secretary of War, the Secretary of the Interior, or the board to carry out the authority hereby conferred in the most economical and efficient manner, he or it is hereby authorized and empowered in the exercise of the powers of national defense in aid of navigation, and in the control of the floodwaters of the Tennessee and Mississippi Rivers, constituting channels of interstate commerce, to exercise the right of eminent domain for all purposes of this act and to condemn all lands, easements, rights-of-way, and other area necessary in order to obtain a site for said Cove Creek Dam, and the flowage rights for the reservoir of water above said dam, * * * When said Cove Creek Dam * * * shall have been completed, the possession, use, and control thereof shall be entrusted to the corporation for use and operation in connection with the general Tennessee Valley project and to promote flood control and navigation in the Tennessee River.

In an amendment to the act, the Congress, in 1935, gave more specific directions for the use of the project as follows:

> SEC. 9a. The board is hereby directed in the operation of any dam or reservoir in its possession and control to regulate the stream flow primarily for

[1] H. Doc. No. 328, 71st Cong., 2d sess.

the purposes of promoting navigation and controlling floods. So far as may be consistent with such purposes, the board is authorized to provide and operate facilities for the generation of electric energy. * * *

In the last study of the United States Army Engineers, five possible locations were chosen at the Cove Creek site for consideration—three upstream and two downstream from the mouth of Cove Creek. Geological formations are similar at all five of the locations considered as well as for any site from Cove Creek upstream to the mouth

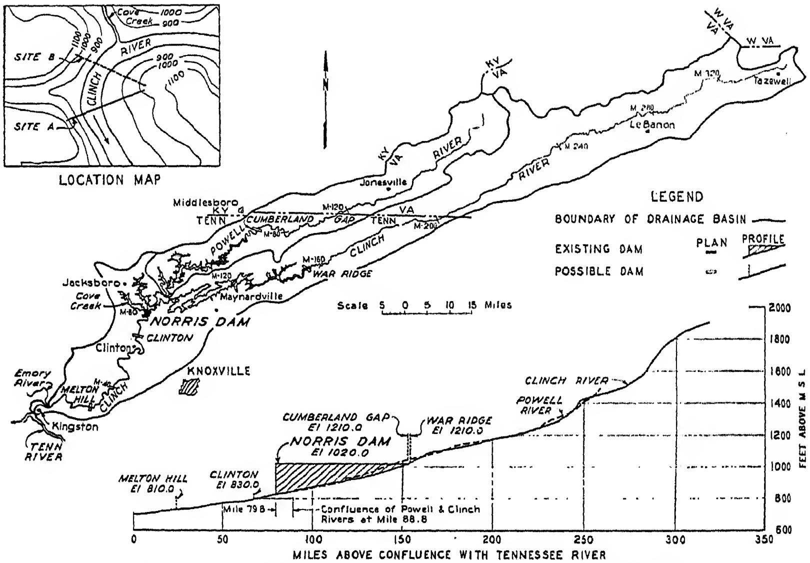


Figure 3.—*Map and profile of the Clinch and Powell Rivers.*

of the Powell River. Because of the advantage of additional water storage in the Cove Creek Valley, choice of the sites was confined to the two locations downstream from Cove Creek, designated as sites A and B (see figure 3). Conditions at site B were somewhat uncertain as examination of the site by drilling was not so extensive as at site A. A consideration of the two sites failed to show any outstanding relative advantage in either, so that site A was chosen because of more definite information available at this point.

Determination of controlling elevations.

Cove Creek Dam, in the United States Army Engineers' plan[2] for navigation, flood control, and power development in the Tennessee River Basin, was to have a normal pool level at elevation 1,050 and a surcharge level at elevation 1,057. A diversion type spillway (see fig. 19) was provided with a crest at elevation 1,032. Ten 18- by 27-foot Stoney gates were to be provided at the spillway. A barge lift

[2] H. Doc. No. 328, 71st Cong., 2d sess.

was included in the design to care for navigation on the Clinch River. The plan provided for the storage of all available run-off from year to year.

The plan provided for a power plant with a capacity of 165,000 kilowatts. This approached the average potential stream capacity for the highest periods in the highest months for 24 years, 1903 to 1926 inclusive. In only one year, 1907, would there have been a substantial waste of water because of the limit of capacity provided. The infrequency of the periods of excess flow and their limited length indicated that the really economic installation for this plant as a unit in the ultimate power system of the general scheme was possibly somewhat lower than recommended above; but as the plant was in a large measure designed to meet emergency requirements over a considerable period of years while the balance of the watershed was being developed, it was advisable that the plant should be such that it could, when necessary, supply something above normal economic demands for power and water. The function in the system was to be confined to sustaining primary power by supplying water to downstream plants as well as by its own power output. Since no sluices were provided in the dam, all water discharge in the reservoir would have been passed through the turbines or over the spillway.

In July 1933, after the passage of the Tennessee Valley Authority Act, a study of the above design was made by an engineering consultant of the United States Army Engineers with the view of determining the most economical height for a dam at the Cove Creek site. This dam differed from the previous design of the United States Army Engineers, being intended for a broader multi-purpose project as specified in the act. This study revealed that within a range of about 40 feet no definite determination of the most economical height of the dam was possible. Further development of navigation and power facilities on the main river and the future power demand to be served would have a direct bearing on the most economical height of the dam. These conditions could not be forecast at the time of the study with sufficient accuracy to determine the height within a 40-foot range which, in the long run, would give the optimum results.

This study recommended a dam with a maximum water level at elevation 1,060 and with upper power pool level at elevation 1,030. The draw-down was to be from elevation 1,030 to 970, between which there was a storage of 1,760,000 acre-feet. The volume between elevation 1,030 and 1,060, 1,480,000 acre-feet, was to be reserved for the detention of flood peaks. The conclusion reached in the study was that the recommended height appeared to be reasonable, both from a geographical and economic standpoint and that it would satisfy the threefold purpose which the project was to serve.

Demands for navigation were considered in the Authority's plan of the Norris project in 1933. At the time of the early studies of the TVA, no information was available regarding the extent of the program which might later be authorized by the Congress. In its original form, the Tennessee Valley Authority Act did not specifically provide for the complete canalization of the Tennessee River to Knoxville, although this appeared to be a major objective in House Document No. 328, and in the River and Harbor Act of 1930. There

was a possibility that Norris Dam, together with Wilson Dam and a series of low dams on certain reaches of the main river, might eventually constitute the entire river development. Consequently the Norris project had to be designed so as to contribute as much as possible toward increasing the low water flow of the river as well as to serve for flood control and power development. The amendment to the act in 1935 provided for the construction of dams and reservoirs which would provide a 9-foot channel from Knoxville to the mouth of the river.

In the Authority's studies it was not found economical to provide for "hold-over" storage from year to year for power because of the increased cost of backwater protection at the higher pool elevations involved. This was especially true because the higher elevations would cause great damage to property at Caryville and La Follette, located on the reservoir. It would have been necessary also, with some of the higher proposed designs, to construct additional saddle dams at low places on the reservoir rim. The feasibility of operation of "hold-over" storage on multi-purpose projects did not appear promising.

The determination of the upper pool elevation was a difficult problem. It was necessary to obtain an economic balance between cost and benefits to navigation, flood control, and power development. In consideration of these factors, together with various related engineering, forestry, land planning, and other problems that had to be considered, all previous investigations made by the various agencies involved in the development of the dam at this site were reviewed and studied in detail. This study resulted in the selection of elevation 1,020 for the spillway crest as the most advantageous. A study of available rainfall and stream flow data indicated that it would be possible to fill the reservoir to this elevation during all but exceptionally dry years, with allowance made for the release and use of the stored water. The dam was designed to be structurally safe with the reservoir level rising to elevation 1,052.[3] The top of the roadway over the dam was fixed at elevation 1,061 in order to give ample clearance under the spillway bridge in the event of the water level reaching the elevation stated. Subsequent studies indicate that water level at elevation 1,034, the top of the spillway gates, would be reached rarely and that even with extremely heavy rainfall and recurring storms, the maximum level will not exceed elevation 1,047.

The spillway capacity required to pass the most extreme flood flows (inflow less detention) was fixed at 200,000 cubic feet per second with the pool at elevation 1,047. A spillway 300 feet in length with the crest at elevation 1,020 will pass this flow. To make available the 14-foot space above the spillway crest for flood control, 100- by 14-foot drum gates were provided. In addition to the spillway discharge of 200,000 cubic feet per second, eight outlet conduits were provided, having a total discharge capacity of 40,000 cubic feet per second (pool at elevation 1,034) for the release of floodwaters and to release water for navigation, flood control, and other draw-down purposes. The capacity of the conduits was limited to 40,000 cubic feet per second because that was approximately the

[3] With the dense concrete secured at Norris Dam, the structure is safe from overturning against a theoretical reservoir level at elevation 1,060.

maximum flow which the Clinch River, below the dam, could normally pass without causing substantial damage by overflow.

The determination of these principal controlling elevations required many detailed studies. It is impossible in this report to describe all of these and to furnish the preliminary data involved; however, some of the more important considerations are summarized in later sections of this chapter.

DETAILED DESCRIPTION OF DAM SITE

At the dam site the river flowed in a southwesterly direction and had a total width between river banks of about 400 feet. Within the limits of the dam, the east abutment rises quite gradually, extending

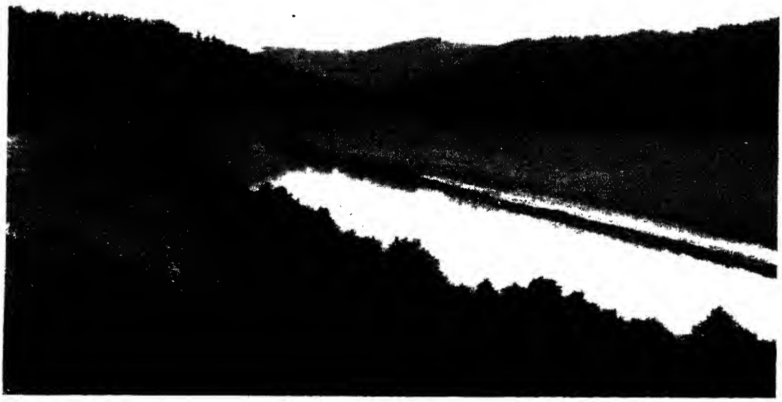


FIGURE 4.—*Site A before construction was started.*

for some 1,200 feet and rising but 240 feet. It is overlain with a heavy overburden of clay, averaging about 60 feet in depth. The west abutment is very steep, extending some 270 feet while rising 240 feet. Very little overburden exists, and the surface rock was exposed and badly weathered. The over-all length of the dam required at location A was 1,860 feet.

Geology of site.

Due to the prominence with which the Cove Creek sites had been viewed for many years, the sites and surroundings had been examined by several geologists prior to the organization of the Authority. When the dam construction became certain, it was evident that a more detailed study was desirable, and therefore, in the summer of 1933, more extensive geologic work was performed under the direction of the Authority. This new work resulted in mapping an area of about 80 square miles to a scale of 1:126,720. In this study the old Knox formation was arranged in six subdivisions because all

the reservoir problems were involved in the lithology of the Knox. In addition to the general areal mapping, special detailed studies were made of the dam foundations and abutments and of the cave and sink systems in the area near the dam.

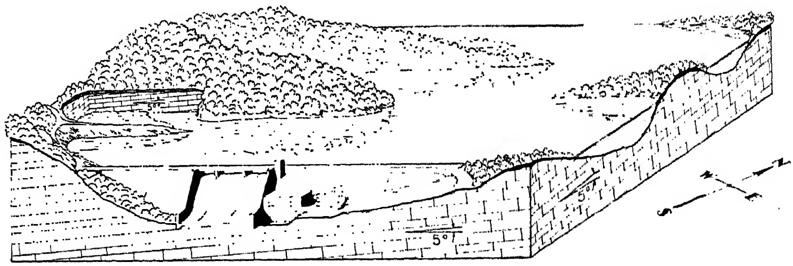


FIGURE 5.—*General geologic formation at dam site.*

In the immediate vicinity of the dam the only series of rock is dolomite, which belongs to the lower part of the Copper Ridge formation and is in turn the lowest part of the old Knox group. The Copper Ridge dolomites are almost exactly dolomitic so far as the percentage of lime, carbonate, and magnesium carbonate are concerned. They contain from five to ten percent silica. The following analysis is of a sample taken from the quarry which was separated from the dam site only by a ravine:

Silica (SiO_2)	5.22
Alumina (Al_2O_3)	0.23
Ferric Oxide (Fe_2O_3)	0.40
Lime (CaO)	29.40
Magnesia (MgO)	20.28
Loss on Ignition 100–1,000° C	44.31

While silica and the proportion of the other minerals listed in the above sample varied somewhat from that shown, the above sample is representative. The silica is concentrated chiefly in beds of chert interspersed with thicker and more numerous beds of practically pure dolomite. Thickness of beds varied from a few inches to as much as 20 feet but both of these extremes are exceptional. The normal or more common beds are those ranging from 6 inches to 5 feet in thickness. The rock exposed on the quarry face corresponds exactly with the series exposed from the top of the right abutment to the base of the dam.

Geology near the dam.

Since the dam site was located on the north side of the Powell River anticline, the normal dip of the rocks is downstream and somewhat into the right bank. There are minor rolls in the strata in other directions, but the average is as indicated in figure 5. The dip rarely is greater than 10°, and in most cases is from 3° to 5°.

There are unusually few faults in this particular portion of the region. It seems probable that the ravine lying on the west bank of

the river between the dam and the quarry site is the site of a fault. That probability is strengthened somewhat by the fact that in line with this ravine, on the left or east abutment, was found a rather continuous fracture zone, which gave rise to somewhat extensive vertical solution cavities. With this exception, faulting or even serious jointing did not in any way hamper the engineering operations at the site chosen.

Condition of foundation rock.

Sound rock was encountered almost immediately below the river bed, and there were surprisingly few solution effects of any sort visible across the foundation.

Condition of abutments.

The soundness of the foundation of the dam contrasts sharply with the condition of both the right and left abutments. The right abutment is of essentially sound rock but is pierced at frequent intervals by vertical solution channels and horizontal caves. On the left abutment conditions were more serious. Here, the weathering had gone so far that the old cave systems were practically destroyed and decayed rock was found in many places almost down to the present river level.

River history and cave levels.

From the very outset it was obvious that there could be no real understanding and no real interpretation of drill records unless some tentative hypothesis as to the later geological history of the Tennessee River Basin was determined. Beginning with such an hypothesis, checking it by the actual results obtained in the very extensive drilling operations and modifying it as necessary, a rather complete history of the later stages of the Tennessee River drainage system was obtained.

The Cretaceous and early Tertiary history of the region furnishes a background for later developments, but the real interest in the problem does not date beyond the Pleistocene and late Tertiary. For this reason it may be pointed out that the pre-Tennessee drainage system, whatever it may have been, does not seriously enter into the geologic field work of the dam site.

All the work in engineering geology at the site is concerned with the very latest Tertiary and the whole Pleistocene ages. The reason for this engineering interest in late geologic history is that the earth movements in any region affect the depth to which weathering, solution, and caves are likely to occur. Under normal conditions, excluding artesian and other special conditions, even limestone rock cannot be seriously attacked by water below the level of a nearby stream valley. If, therefore, the land did not change in elevation, weathering, caves, and other solution effects would be expected to cease at a very short depth below the bed of the present major stream of the region—in this case, the Clinch River. If, on the other hand, land had at one time been elevated much above the present level, the weathering of that period would have been extended to depths far below present stream levels.

There seems to be conclusive evidence that there has been a series of uplifts of the land with practically no depression. The evidence for this lies in the existence of several distinct series of cave levels all over the region. Where best developed, there are about three distinct series of caves located at elevations approximately 100 feet apart. Each of these cave levels marks a point at which the river stood for a long time in its history—long enough to develop a cave series leading into the river at that level. After each of these cave forming periods, the land rose again approximately 100 feet, and a new series of caves with associated underground drainage was developed to fit the new river level. This has happened at least three times in the region. The present era constitutes a fourth period and in the course of thousands of years a new cave system will probably develop at about the present level of the river.

The cave series are best shown on the right, or west, abutment where they involved considerable engineering work. The caves are all in the purer, and therefore the more soluble limestone beds of the series. Water passes the chert beds through vertical cracks, but cannot dissolve the chert sufficiently to form caves in it. Each cave series, therefore, consists of a practically horizontal cave dissolved out of the rather pure limestone and connected with the surface, or with an upper cave level, by a number of narrow vertical channels or cracks.

Necessity of foundation treatment.

Preliminary core borings, made prior to actual construction and during the early part of the excavation period, indicated that some foundation treatment would be necessary. In the main foundation area it was found that the seams between the horizontal bedding planes, while not very wide, were filled with clay and other residual material which was sufficiently porous to permit leakage of water underneath the structure. It was therefore decided that these seams should be washed free of this foreign material and thereafter grouted. At other points in the foundation, grouting was carried on not as a matter of actual necessity, but rather to remove possible uncertainties. As a sharp contrast to the soundness of the foundation of the dam itself, the abutment areas were pierced at frequent intervals by the vertical solution channels and horizontal caves previously mentioned. In the right abutment these required extensive treatment. This treatment was to be accomplished by blocking the caves with concrete and by grouting both the caves and channels through drill holes. On the left abutment, conditions were more simple but also much more serious. A core wall was constructed in the earth embankment. Existing channels and caves were concreted and grouted as a separate operation.

TABLE 1.—*Results of tests on 4⅜-inch diameter drill cores from Norris Dam foundation*

Dry state:	
Ultimate compressive stress, pounds per square inch	29, 300
Modulus of elasticity:	
3,000 pounds per square inch	5, 400, 000
10,000 pounds per square inch	7, 500, 000

TABLE 1.—*Results of tests on 4⅜-inch diameter drill cores from Norris Dam foundation*—Continued

Dry state—Continued.	
Poisson's ratio:	
3,000 pounds per square inch	0.11
10,000 pounds per square inch	0.18
Specific gravity	2.76
Soaked 25 days:	
Ultimate compressive stress, pounds per square inch	26,550
Modulus of elasticity:	
3,000 pounds per square inch	4,100,000
10,000 pounds per square inch	6,600,000
Poisson's ratio:	
3,000 pounds per square inch	0.09
10,000 pounds per square inch	0.16
Percent absorption by weight	0.52
Under 690-foot head for 7 days:	
Ultimate compressive strength, pounds per square inch	27,700
Modulus of elasticity:	
3,000 pounds per square inch	4,500,000
10,000 pounds per square inch	6,900,000
Poisson's ratio:	
3,000 pounds per square inch	0.12
10,000 pounds per square inch	0.17
Percent absorption by weight	0.40

Test on foundation rock.

Shot drill cores, 4⅜ inches and 17 inches in diameter, were taken from the rock approximately at right angles to the bedding planes to determine the physical characteristics of the foundation rock. Determinations were made of specific gravity, absorption, saturation under high hydrostatic head, elasticity, and compressive strength. The results of these tests [4] as well as a comparison with test results on rock from other dam foundations are given in tables 1 and 2. From the results and comparisons of the tests it was found that both the compressive strength and the elastic values were sufficiently high to indicate that the foundation rock for the dam was adequate to withstand the normal pressures to which it might be subjected.

TABLE 2.—*Comparison of test results—foundation rock, Norris, Boulder, and Grand Coulee Dams*

	Boulder	Grand Coulee	Norris
Type of Rock	Breccia.	Granite.	Dolomite.
Size of core, inches	4¾ x 9½	4¾ x 9½	4⅜ x 8¾.
Number of cores tested	16	24	15.
Ultimate compression stress, pounds per square inch:			
Maximum	22,100.	29,700	37,100.
Minimum	9,260.	5,280.	17,200.
Average	15,920.	15,500	28,150.
Modulus of Elasticity, pounds per square inch:			
Maximum	7.5×10^6	7.6×10^6	$7.2 \times 10.^6$
Minimum	4.9×10^6	0.6×10^6	$3.4 \times 10.^6$
Average	6.0×10^6	3.1×10^6	$5.4 \times 10.^6$
Poisson's Ratio, pounds per square inch:			
Maximum	0.38.	0.21	0.14.
Minimum	0.19.	0.07	0.06.
Average	0.24.	0.13	0.11.
Stress in pounds per square inch of secant elastic values	2,000 to 3,000	1,000 to 2,800	3,000.

[4] U. S. Bureau of Reclamation Technical Memorandum No. 481.

COLLECTION OF DATA

Rainfall data.

The watershed of the reservoir contains 2,912 square miles, most of which is in the form of steep-sided, narrow valleys. At the outset of the project there were but four United States Weather Bureau rain gages in the area and a few nearby in adjoining watersheds. The Authority has established additional daily rain gages and several recording gages, so located that the integrated rainfall over the entire drainage area can be accurately computed.

The gages are located at intervals of about 15 or 20 miles, some being placed at high altitudes and others in river valleys. In all cases consideration was given to the location of the gage with respect to topography in order to obtain representative data. These gages were of substantial use in predicting floods during the construction period and will be of continued use in furnishing hydrological data for water control operations. The information thus obtained is compiled in the form of daily, monthly, and annual totals for the individual stations; isohyetal maps of the area are prepared for monthly periods or for the more important individual storms.

While many of the United States Weather Bureau stations have continuous records of rainfall extending over periods of 60 years or more, it was desired to obtain records covering a longer time. Dendrochronological studies were therefore made upon samples from a number of trees in the Clinch-Powell area. This study, involving the measurement of about 40,000 tree rings, resulted in a tree-growth curve extending over a period of approximately 300 years. The comparison of this curve with available rainfall records indicates that the correlation between the two is close.

Flood investigations.

A knowledge of past flood history is necessary to a flood control project to determine spillway discharge and storage capacity for future floods and to develop schedules for operating the reservoir. Inquiries and investigations of old high water marks were initiated by the United States Army Engineers at the time of the original survey for the proposed Cove Creek Dam. These have been carried on by the Authority to the extent that high water marks have been obtained for different floods from 1826 to date.

Flood Control, Navigation, and Power Studies

The Clinch River Basin.

The Clinch River above the dam, draining an area of 2,912 square miles, has a length of nearly 300 miles, more than half of which is within the State of Virginia. Its main tributary, the Powell River, has a length of about 180 miles and a drainage area of 938 square miles. The Clinch River flows between ridges from the northeast to the southwest and the valley has a maximum width of about 15 miles. The Powell River occupies a narrow basin parallel to that of the Clinch River and also flows between mountain ridges for practically its entire length.

The rugged topography and steep slopes favor a quick run-off. The elongated shape of the basin, however, tends to delay the concentration of flood flows. The area is within a limestone region in which caves and sinkholes are common, and solution channels in the limestone provide for underground flow. In some cases these underground channels return to the surface and contribute to the surface water flows. The average rainfall computed for a 30-year period is about 48.5 inches, ranging from about 40 inches at the headwaters in Virginia to 52 inches at the dam.

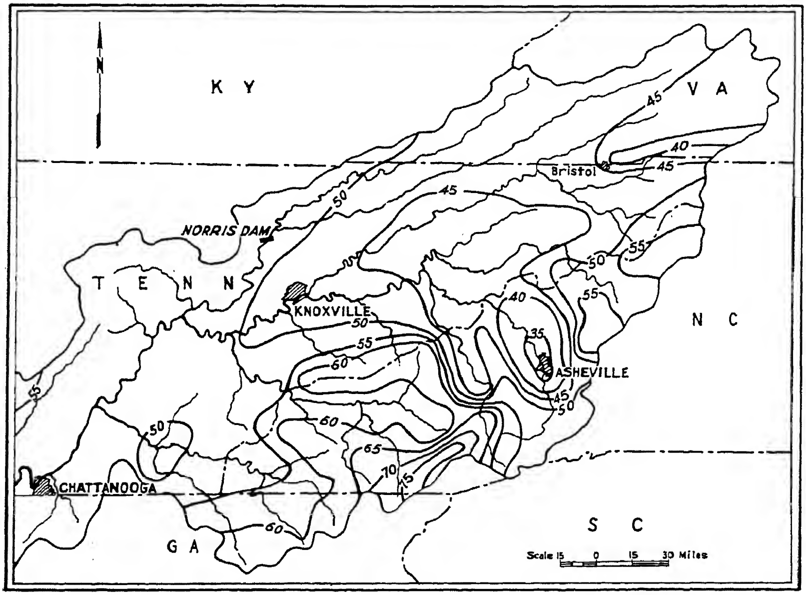


FIGURE 6.—*Average annual rainfall, eastern portion of the Tennessee Valley.*

Importance of flood control.

Damaging floods are more serious and of much more frequent occurrence in the upper Tennessee Basin than in the lower basin. The city that has suffered most from floods is Chattanooga, located at the dividing point between the upper and lower basins. The flood stage at Chattanooga has been exceeded over 50 times in the 70 years for which records are available. Although past damage has been very severe, the larger floods occurred when the city was very small in comparison with the present population. A repetition of the flood of 1867, the largest on record but not the largest that could be expected, would be disastrous to the city, affecting practically all its industrial area, the more important business area, as well as a large part of its residential area. Railroad communication and highway communication would be cut off, and a serious loss of life might be involved

The area covered by the present city of Dayton, Tenn., was nearly submerged in the 1867 flood, and other communities along the river would have been substantially damaged if they had existed at that time. The Clinch River influences the Tennessee River flow at all points below Kingston.

Although the drainage area above the dam is very small in comparison with the Ohio River drainage area at Cairo, the Clinch River flows have not been without influence on the flood crests at Cairo.

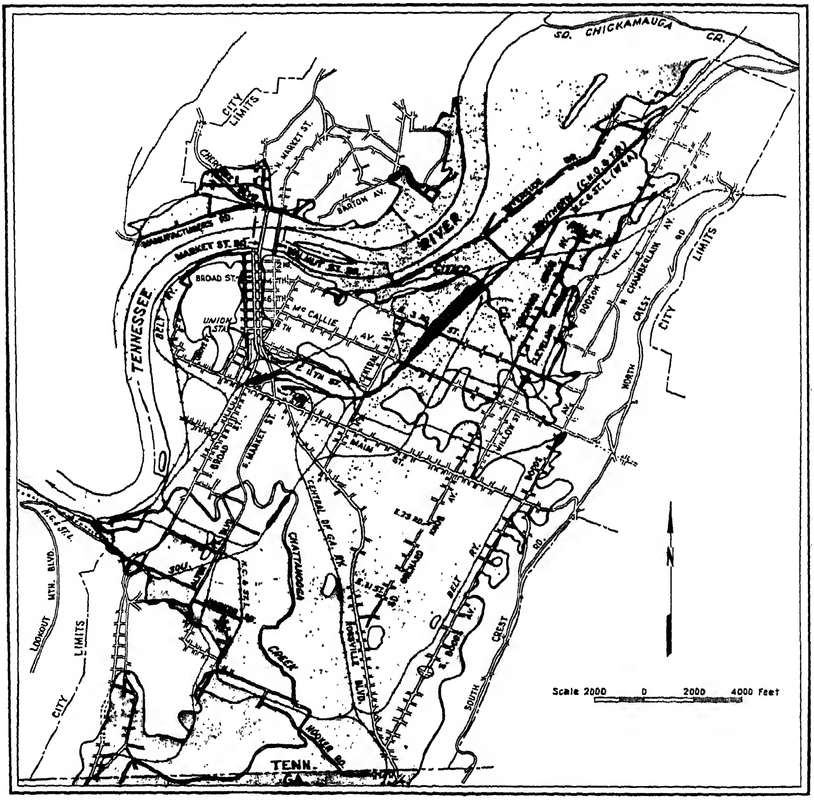


Figure 7.—*Area of flooding at Chattanooga in the 1867 flood.*

A flow of 30,000 cubic feet per second at Clinton has not been a rare occurrence. Such a flow at the crest of the higher floods at Cairo accounts for about 6 inches of the flood stage. This difference is of substantial importance in cases when it was necessary to increase the margin of safety in the flood wall at Cairo. The flood of January and February 1937, which occurred after Norris was completed and in operation, showed the value of even small reductions in flood stages at this point.

Past floods.

An indication of the magnitude and occurrence of past floods on the Clinch River and on the Tennessee River is given in table 3.

TABLE 3.—*Number of times that stated river stages have been exceeded at Clinton and Chattanooga*

Stage, feet	Clinch River at Clinton (flood stage 25 feet), 1885 to 1903	Tennessee River at Chattanooga (flood stage 30 feet), 1867 to 1936
25	42	---
30	15	55
35	5	29
40	1	9
45	1	4
50	---	3

NOTE.—Stage records are incomplete at Clinton between 1885 and 1897, and at Chattanooga between 1867 and 1875.

Although the population in the valley was small in 1867 and even Chattanooga had only about 5,000 inhabitants, the newspapers of the day report a great disaster involving loss of life and much suffering. The area of flooding at Chattanooga in that flood is shown on figure 7; the depth of flooding exceeded 25 feet over a substantial area.

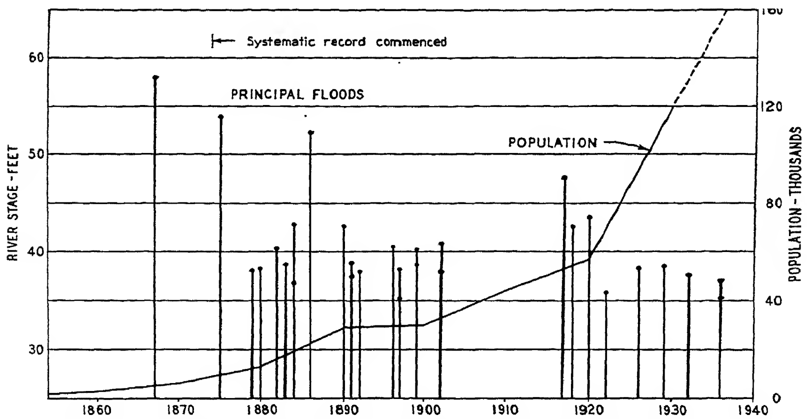


FIGURE 8.—*Occurrence of floods at Chattanooga and the population growth curve.*

The occurrence of the principal floods at Chattanooga and the population curve, according to the United States Census, are shown in figure 8. It will be noticed that the floods of 1875, 1886, and 1917, the last mentioned reaching a stage of 47.7 feet, were successively lower in stage. There is no apparent reason for this progressive decrease in the magnitude of successive floods. Changes have occurred in the river basin, but these have been such as to cause, according to popular belief, an increase rather than a decrease in flood magnitude and frequency.

Contribution of Clinch River to Chattanooga floods.[5]

The Tennessee River at Chattanooga has a drainage area of 21,400 square miles, which includes the basins of the five main tributaries: the Clinch, Holston, French Broad, Little Tennessee, and Hiwassee Rivers. The drainage area for the Clinch River above Norris Dam is about 14 percent of that of the Tennessee River above Chattanooga, but the Clinch's contribution to Chattanooga floods has been generally a larger percentage of the peak flow. A comparison of flood peaks at Clinton on the Clinch River with those on the Tennessee at Chattanooga is shown in figure 9.

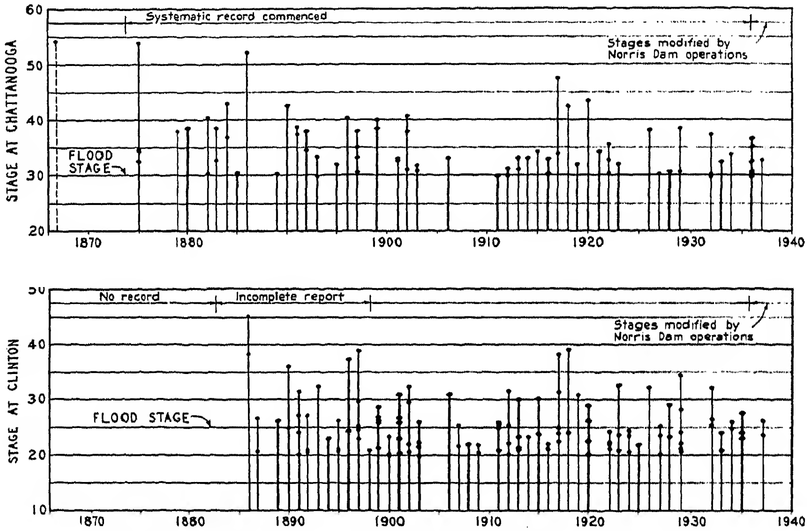


FIGURE 9.—*Comparison of flood peaks at Clinton and Chattanooga.*

TABLE 4.—*Contribution of Clinch River to Tennessee River floods at Chattanooga*

Date	Tennessee River at Chattanooga: Discharge, cubic feet per second	Tennessee River at Chattanooga: Stage, feet	Clinch River at Clinton, discharge cubic feet per second
Apr. 3, 1886	391,000	52.2	91,000
Mar. 2, 1890	294,000	42.6	59,000
Mar. 11, 1891	264,000	38.9	34,000
Apr. 5, 1896	276,000	40.5	76,800
Feb. 9, 1899	261,000	38.6	46,700
Mar. 22, 1899	274,000	40.2	24,000
Jan. 2, 1902	279,000	40.8	54,000
Mar. 7, 1917	341,000	47.7	60,600
Feb. 1, 1918	289,000	42.7	54,400
Apr. 5, 1920	298,000	43.6	21,000
Dec. 31, 1932	240,000	37.6	20,600

The flow given for the Clinch River is the estimated flow from this stream reaching Chattanooga at the time of the crest.

[5] The Chattanooga flood problem is discussed in detail in H. Doc. No. 91, 76th Cong., 1st sess.

Table 4 shows the esti ... er for the period during which reco... ...ailable.

It may be noticed that in the 1886 flood, the third in magnitude at Chattanooga, there was a very heavy contribution from the Clinch River. No information is available as to the flow contributed in the larger previous floods of 1867 and 1875. The Clinch River in 1886 reached a stage 20 feet above flood stage.

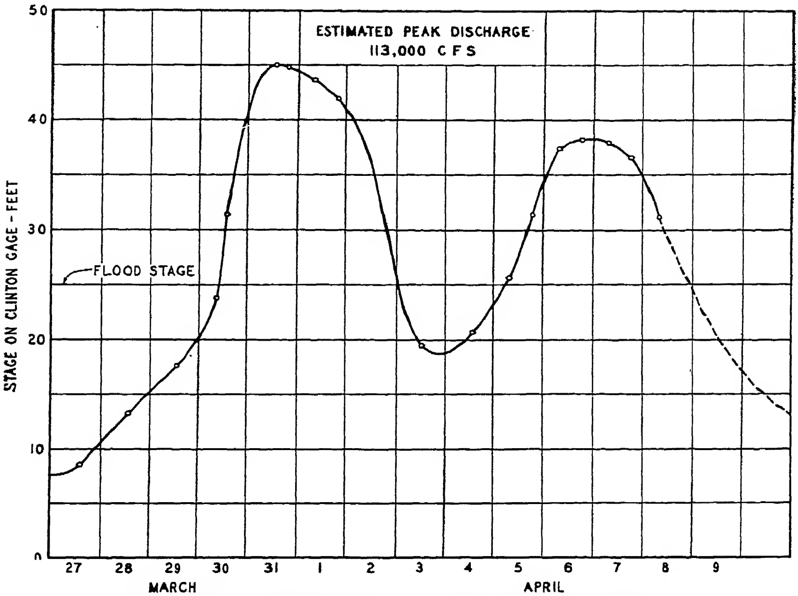


FIGURE 10.—*Maximum flood of record at Clinton (1886).*

Control of the Clinch River at Norris Dam is important not only because of the substantial flows the river contributes but also because the time of travel is such that the peak of the flood wave on the Clinch River reaches Chattanooga near the time of the peak at that city. It is estimated that if the flood flow of the Clinch River had been withheld at the time of the Tennessee River peak, the Chattanooga stage in the 1886 flood would have been reduced from 52.2 feet to 43.8 feet. In the flood of 1917, the fourth flood in order of magnitude, the stage would have been reduced from 47.7 feet to 41.8 feet.

Intense rainfall storms.

Although extreme local storms have in a few cases caused a moderate rise above flood stage in the river at Chattanooga, all the large floods at that city have been caused by general storms covering practically the entire basin. The storms which have produced heavy rainfall over large areas within the Tennessee Basin and those storms on other areas which might in the future be duplicated by similar storms over the Tennessee Basin are of special ...

Extreme rates of rainfall have resulted from storms over the States bordering the Gulf of Mexico and those along the South Atlantic Coast; but as such storms lose their greatest intensity before entering the Tennessee Valley, the rainfall recorded over areas near the Gulf or Atlantic Ocean has limited significance with respect to flood rainfall in the Tennessee Basin.

Unfortunately there is slight information about the rainfall which caused the 1867 flood. Some data are available about the storms causing the floods of 1875 and 1886, although there are not sufficient data for a complete study of either of these floods. In the storm causing the flood of 1886, the line of maximum rainfall entered Tennessee near the boundary line between Alabama and Georgia and extended northeasterly near the northwesterly boundary of the Tennessee Basin. This storm was such as to produce very heavy rainfall over the Clinch River Basin. The line of heaviest rainfall in March 1897 is of interest because it passed more nearly along the axis of the Tennessee Basin than any of the other major flood storms. This storm caused substantial flood heights in the upper basin and the maximum recorded flood in the lower basin. The line of this storm entered the basin near the northeasterly corner of the Mississippi and extended across the State of Tennessee into Virginia, lying within the Tennessee Basin for nearly its entire length. The flood of March 1917 was caused by a storm which entered the Tennessee Basin from Georgia, crossing the southwesterly corner of North Carolina and running a more northerly course than the previous storms.

There were moderate floods in the Tennessee Basin in December 1926, and January 1927, caused by a storm which was more nearly centered over the Cumberland River. The line of maximum rainfall crossed the Tennessee River in the eastern part of the State about midway between the southern and northern boundary line. It then passed along a line approximately parallel with the northerly Tennessee line, causing substantial rain throughout the State. A report of this storm stated:

> It can probably be safely said that in 56 years of Weather Bureau records no other period of nine days will show an equal amount of rainfall over as wide an area in Tennessee. This essentially and primarily was the cause of the unprecedented flood of December 1926.[6]

"The unprecedented flood" referred to the Cumberland River Basin as that flood established a new record along that river. Very high flood rates were recorded on some of the Tennessee River tributaries; and if the storm had been located along a more southerly course, much more serious floods would have occurred in the Tennessee Basin. The rainfall totaled between 10 and 12 inches over an area equivalent to more than one-third of the area of the State of Tennessee.

In March 1929, there occurred a storm which produced what was said to be "in many respects the most disastrous that has yet visited Tennessee."[6] It was a storm of extreme intensity over a short period. The path of this storm extended from northern Alabama across the State of Tennessee into Kentucky. The heavy rainfall

[6] Division of Geology, State of Tennessee, Bulletin 40. Surface Waters of Tennessee.

was divided between the Cumberland and Tennessee Basins, but principally over the Cumberland Basin. It produced flood rates on the Emory River, a tributary entering the Clinch River a short distance from its mouth, and extreme rates on upper streams of the Cumberland River Basin. Some of these rates were outstanding in comparison with other flood records in the eastern United States. The rainfall along the axis of the storm ranged from about 6 to 11 inches. At some points near the center practically the entire amount

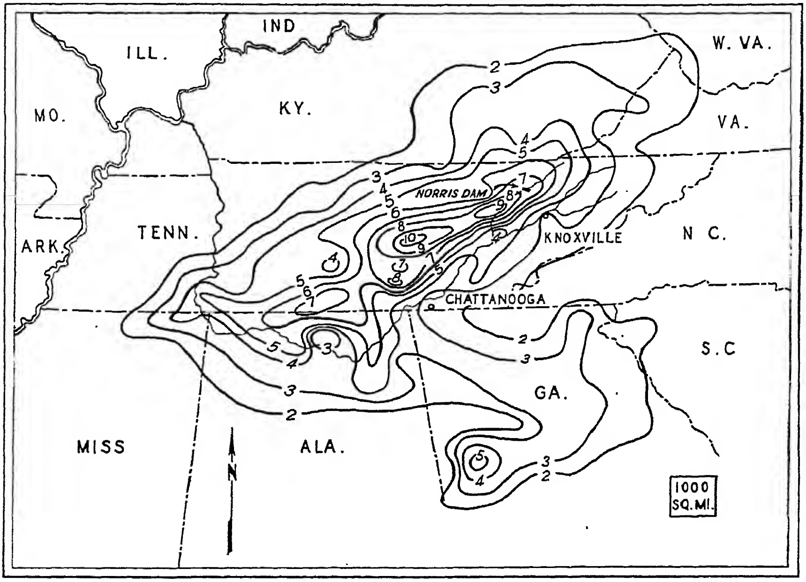


Figure 11.—*Storm of March 1929.*

fell within a period of about 15 hours. It is believed that this storm is typical of one which may visit other parts of the upper Tennessee Basin so as to produce extreme flood rates over substantial areas. The area covered by the storm in its actual location, however, was not sufficient to produce a maximum flood from the entire upper basin. A significant feature of this occurrence was the extreme rate of run-off. From careful observation made directly after the flood by the United States Geological Survey upon both rainfall and run-off, it was estimated that the run-off over very substantial areas was between 90 and 97 percent of the rainfall. (See table 5.)

In November 1906 there was a very heavy storm extending near the valley of the Mississippi and Ohio Rivers from Arkansas to Ohio. Very heavy rainfalls occurred over extensive areas. However, as it was close to the main streams and occurred in a season when conditions were not well suited for a high rate of run-off, no great damage was caused.

TABLE 5.—*Rainfall and run-off of selected streams adjacent to the Clinch Basin during the flood of March 23–31, 1929* [1]

River	Location	Drainage area (square miles)	Average rainfall above (inches)	Total rainfall (acre-feet)	Total run-off (acre-feet)	Run-off (percentage of rainfall)
Cumberland	Barbourville	982	4.53	238,000	226,000	95.0
Do	Cumberland Falls	2,010	4.90	525,000	505,000	96.0
Do	Burnside	4,890	5.00	1,300,000	1,190,000	91.5
Rockcastle	Rockcastle Springs	746	4.20	162,000	150,000	93.0
New River	New River	312	8.00	133,000	125,000	94.5
South Fork Cumberland.	Nevelsville	1,260	6.06	408,000	380,000	94.6
Collins	McMinnville	624	7.50	249,000	226,000	91.0
Obey	Byrdstown	416	5.28	117,000	114,000	97.5
Stones	Smyrna	552	5.90	174,000	165,000	95.0
Caney Fork	Rock Island	1,640	8.45	740,000	694,000	93.7
Do	Silver Point	2,100	8.39	938,000	912,000	97.3
Emery	Harriman	793	9.00	324,000	307,000	

[1] Division of Geology, State of Tennessee, Bulletin 40, Surface Waters of Tennessee.

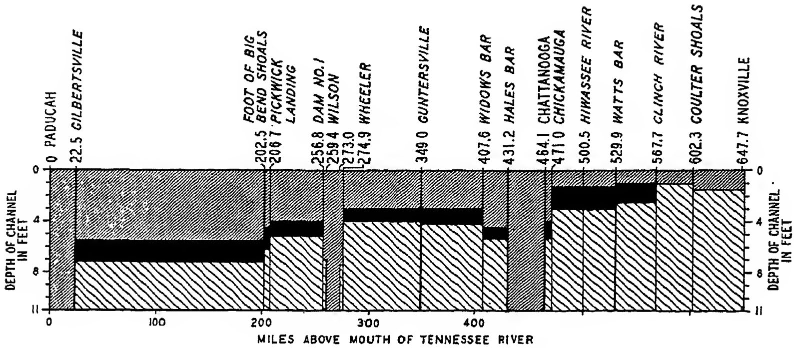
Prior to the January 1937 storm, probably the most notable flood storm over the Ohio Valley was that of March 1913 which passed from Missouri to New York, affecting the northern tributaries of the Ohio River. Consideration has been given to the effect of this storm passing along a line such as to bring the greatest intensity of rainfall within the Tennessee Basin. It is estimated that a maximum upper Tennessee River flood of the future is likely to be caused by such an occurrence. This was a general storm covering a wide area and was of the type that would produce a general flood of greatest magnitude at Chattanooga. Estimates of the magnitude of the future maximum flood that should be anticipated at Chattanooga are predicated on the occurrence of such a flood as this passing along a line of critical importance within the upper Tennessee Basin. Such a storm occurred in January 1937 after the dam was completed and substantial rains fell over the entire Tennessee Basin. This storm extended from Arkansas to Pennsylvania, affecting principally the southern tributaries of the Ohio River. The 1937 storm produced extremely heavy rains along the lower Tennessee River but over an area where the Tennessee Basin has limited width.

Multiple purpose of Norris Reservoir.

A reservoir for the sole purpose of irrigation, water supply, or power development generally will be filled as rapidly as the stream flow will permit and will be kept full except as the stored water is drawn upon for use. A reservoir for the sole purpose of flood control will be kept empty except during the flood period when the temporary retention of floodwater is necessary. Norris is not solely a local flood protection project. It is a multiple-purpose flood control project. Furthermore, it must be used to increase navigable depths in the Tennessee River, particularly in the lower reaches of the Tennessee during the interim preceding the final completion of the standard channel for 9-foot navigation. Immediately after the completion of Norris, substantial releases from Norris during the periods of low water on the lower river added about 2 feet to the controlling depth of the 250-mile stretch of the river between Wilson Dam and the mouth of the river. Figure 12 shows the controlling depth in the Tennessee River

before the completion of Norris Reservoir and the increase in controlling depths resulting from Norris releases.

Norris Lake may in the future be used for commercial navigation, but its more immediate purpose is to increase during dry seasons the navigation depths for those reaches along the Tennessee River where the depth is inadequate and canalization has not yet been provided. Although the act did not specifically require a benefit to navigation along the Mississippi River, the release of stored water during dry seasons will increase navigation depths in shallow reaches below Cairo.



LEGEND

CONTROLLING CHANNEL DEPTHS PRIOR TO T V A

*INCREASE IN CONTROLLING DEPTHS DUE TO NORRIS RELEASES BEFORE CANALIZATION

DEFICIENCY IN CONTROLLING DEPTHS IMMEDIATELY AFTER COMPLETION OF NORRIS

* Controlling depths greater than those shown were secured in some reaches for brief periods to meet special navigation requirements during the low water season by additional releases from Norris.

NOTE:
Dimensions indicate minimum depths below water surface
11' depths provide for 9' draft

REACH OF RIVER (IN MILES)	CONTROLLING DEPTH 1933 (IN FEET)	CONTROLLING DEPTH AFTER NORRIS (IN FEET)
0 TO 22.5	11.0	11.0
22.5 TO 202.5	5.5	7.2
202.5 TO 206.7	4.5	6.2
206.7 TO 256.8	4.0	5.1
256.8 TO 259.4	7.0	7.0
259.4 TO 273.0	11.0	11.0
273.0 TO 274.9	7.5	7.5
274.9 TO 349.0	3.0	4.0
349.0 TO 407.6	3.0	4.2
407.6 TO 431.2	4.5	5.3
431.2 TO 464.1	11.0	11.0
464.1 TO 471.0	4.0	5.3
471.0 TO 529.9	1.3	3.0
529.9 TO 567.7	1.0	2.5

FIGURE 12.—*Controlling depths in the Tennessee River before and immediately after the completion of Norris.*

The increase in navigation depths on the Tennessee and streams below the dam will be due to the collection of stream flow during the flood season and its release during subsequent dry seasons. The purposes of navigation would be favored by collecting and retaining as much of the floodwater as possible during the winter season.

As far as may be consistent with the requirements for navigation and flood control the Board is authorized to provide and operate the facilities for the generation of electric energy. The dams and reservoirs therefore should be multiple-purpose projects.

Reservoir operation for power requires the storage of water during the wet season and its release during the periods of natural low stream flow. In this way the operation of the reservoir for power development would be similar to that for navigation.

Navigation, flood control, and power are the most prominent uses of Norris Reservoir. These resources may be utilized for national defense in case of war emergency. Facilities for recreation and other desirable features are also provided.

Flood period.

A fundamental feature in the multiple use of Norris Reservoir is the extent and period for which the use for flood control is essential. During critical periods of general floods on the Tennessee, Ohio, and Mississippi Rivers, it may be desirable or necessary to retain the entire Clinch River flow above the dam. Such floods are known generally to occur during the winter and early spring. At the time

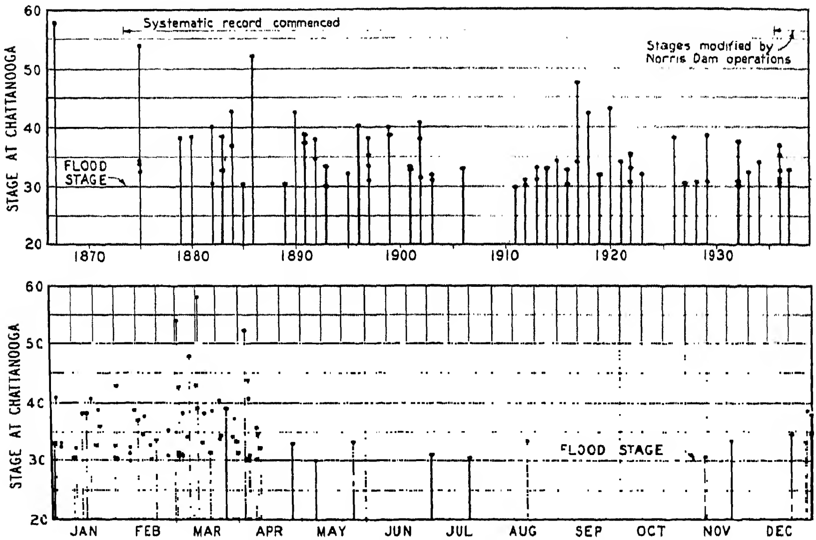


FIGURE 13.—*Occurrence of all record floods at Chattanooga, Tenn.*

of extreme local floods, caused by cloudburst type of rainfall, general floods will not occur; and it will not be necessary to exercise as much control over the flood flow. At such times, flow approximately equal to the bankfull capacity of Clinch River may be released, and such an outflow will very substantially reduce the requirements for storage. Local floods of this kind are likely to occur during almost any season of the year. One very important consideration is the restriction on the release of floodwater after it has been stored for flood control. This difficulty is due to the danger of a repetition of flood producing storms on the Tennessee Basin or the existence of high stages on the Ohio or Mississippi Rivers at the time when release would otherwise be desirable. Ohio River floods might be caused by storms independent of those directly affecting the upper Tennessee Basin.

These considerations necessitate an investigation as to the period of flood danger to ascertain to what extent general floods are limited in their seasonal distribution. Continuous river gage records have been kept at Chattanooga since 1875. There are also records of extreme stages previous to this, including the 1867 flood, which was known to have been substantially greater than any within the preceding 20 years. The occurrence of all record floods at Chattanooga, both with respect to yearly distribution and seasonal distribution, is shown on figure 13, and a similar record for the Clinch River is shown on figure 14. This diagram shows a very definite flood season beginning about the last of December and ending in the early part of April. No flood has occurred causing serious damage at Chattanooga after the first week in April. On the basis of this information, it has been concluded that large flood storage capacity at Norris Reservoir will not be required during the period between April 15 and December 15.

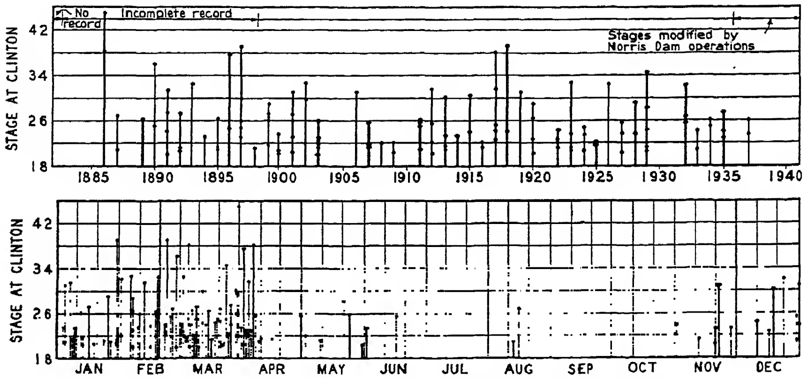


FIGURE 14.—*Occurrence of all record floods at Clinton, Tenn.*

It is recognized that the retention of flood flows at Norris Reservoir may be desired a little later than April 15 in order to reduce Mississippi River stages after the growing season has commenced in the lower Alluvial Valley. No difficulty is anticipated, however, in supplying this need as Norris Reservoir above the spillway level will have capacity to provide efficient control for such flood flows as may require control between April 15 and the early summer.

Reservoir filling.

In adopting any method for filling the reservoir, consideration must be given both to the demands for flood control and to the amount of dependable stream flow available. Storage of floodwater for the protection of Chattanooga will be required at frequent intervals, as the flood stage at that city has been exceeded over 50 times in the record period of 70 years.

One of the difficulties in the operation of Norris Reservoir is due to the danger of repetition of floods within short intervals. In the

early part of the flood season, large capacity is needed to provide against a conflict in the demands for flood control in the upper Tennessee Basin on the one hand and the Ohio and Mississippi Rivers on the other hand and to provide for a possible repetition of floods. As the flood season advances, the chance for repeated floods and the probable need for prolonged storage diminishes. Near the end of the flood season, reservation must be made for the occurrence of a single heavy flood requiring control for Chattanooga.

TABLE 6.—*Run-off from Clinch River Basin at Clinton during flood periods 1898–1937*

[Depth in inches per square mile]

Year	Month					
	Preceding December	January	February	March	April	May
1898	0.50	2.68	0.71	1.06	2.40	1.06
1899	1.00	2.95	5.05	7.25	2.62	2.29
1900	.37	1.16	2.49	3.34	1.07	.33
1901	1.31	2.03	1.26	1.57	4.72	2.87
1902	4.48	3.18	2.54	6.85	1.90	.68
1903	1.74	1.54	4.43	5.28	4.54	.85
1904	.63	.97	1.05	2.90	.98	1.12
1905	1.37	1.81	3.00	2.33	1.24	2.04
1906	1.53	2.44	1.10	1.84	1.95	1.16
1907	1.86	2.20	1.90	3.92	2.47	1.79
1908	1.58	3.17	3.29	3.55	2.69	1.44
1909	2.04	2.71	3.60	4.29	1.91	2.86
1910	.38	1.97	1.75	1.56	1.17	2.14
1911	.84	2.84	3.60	3.07	4.70	1.63
1912	1.86	2.18	2.75	5.48	5.52	2.88
1913	1.34	6.34	2.17	5.52	1.44	1.67
1914	.89	1.01	1.94	2.19	4.47	1.10
1915	2.59	4.04	2.50	1.44	.91	.91
1916	4.37	5.00	2.89	2.43	1.86	.75
1917	1.80	5.56	3.87	10.08	2.08	.81
1918	.45	4.47	2.99	1.96	2.61	1.73
1919	2.20	4.40	1.53	3.06	1.35	2.61
1920	2.99	3.12	3.20	4.92	3.62	1.19
1921	3.00	2.06	3.34	1.54	2.02	1.40
1922	2.71	3.55	3.31	6.09	2.89	3.07
1923	2.12	3.52	5.76	5.11	1.87	2.51
1924	1.64	4.67	2.60	2.44	2.23	2.78
1925	2.52	3.92	2.86	1.31	1.04	1.03
1926	1.03	2.21	3.13	2.04	1.81	1.20
1927	7.64	2.24	4.96	3.14	3.24	1.70
1928	1.73	1.86	1.65	2.57	4.15	2.56
1929	1.24	3.82	2.55	7.24	1.70	5.73
1930	1.61	1.40	2.24	2.51	1.22	1.19
1931	.61	.93	1.10	1.19	3.18	1.52
1932	2.06	2.90	5.90	2.71	2.54	1.94
1933	2.68	3.72	4.80	3.25	1.74	2.12
1934	.57	1.44	.94	6.41	2.30	.67
1935	.87	2.79	2.17	6.02	4.50	2.76
1936	.71	4.30	3.16	4.40	3.94	.48
1937	1.94	7.95	3.84	1.71	1.12	1.15

The stream flow of the Clinch River at Clinton [7] is shown in table 6 for the period December to May, inclusive. The drainage area at Clinton is about 3,056 square miles or only 4.9 percent greater than that at Norris Dam; but as the stream flow is given in terms of inches depth per square mile, the information is applicable to Norris Reservoir.

The total run-off during the flood period in the 1867, 1875, or 1886 floods is not known, although the flood stages at Clinton in the last-

[7] The gage records since 1930 were from the Coal Creek Gaging Station.

mentioned flood are available for most of the flood period. In the month of March 1917, there was a run-off of 10.08 inches. This was the year in which the Chattanooga flood of fourth largest magnitude occurred. The next highest for a single month was in January 1937 when there was a run-off of 7.95 inches. There was a run-off of 7.25 inches in March 1897 and 7.24 inches in March 1929. The maximum run-off in two consecutive months occurred in February and March 1917 when there was a total of 13.95 inches. In January and February 1937 there was a total of 11.79 inches; in February and March 1899 there was a total of 12.30 inches of run-off. The maximum for three consecutive months occurred in January, February, and March, 1917 when there was a total run-off of 19.51 inches.

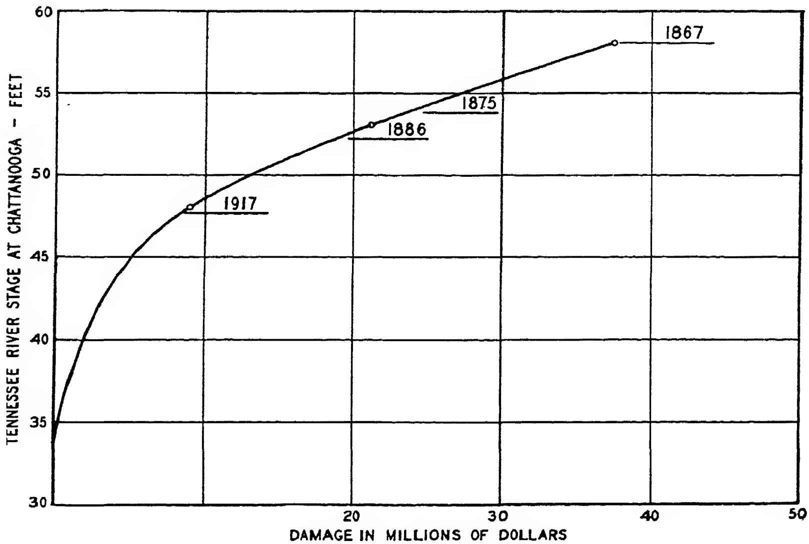


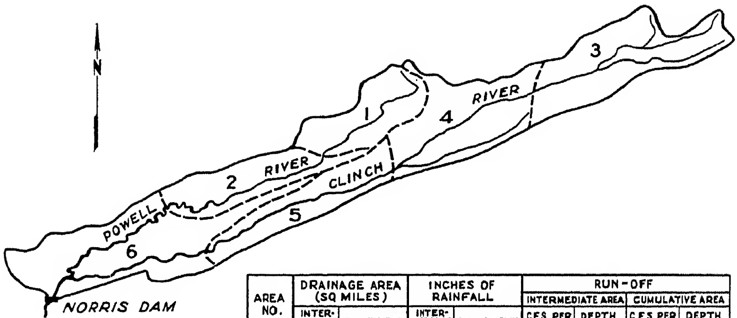
FIGURE 15.—*Potential flood damage in 1938 to city of Chattanooga.*

In January, February, and March of 1899 there was a total of 15.25 inches; and in January, February, and March, 1923, there was a total of 14.39 inches. The largest total run-off for 4 consecutive months was also in 1917; the total for January, February, March, and April of that year being 21.59 inches. The next highest for a 4 months' period was from January through April 1899, when there was a total of 17.87 inches. These figures are of limited significance because the necessity for flood storage is due to high rates of stream flow for flood periods of less than a month. In general, however, the month in which a substantial flood occurs has a large total rainfall. There were substantial floods in a number of the months just mentioned.

The capacity of Norris Reservoir between elevation 955 and elevation 1,045, the highest elevation expected to be reached for flood control, is shown in table 7.

TABLE 7.—*Capacity of reservoir between elevation 955 and elevation 1,045*

Elevation	Volume, acre-feet	Accumulated volume, acre-feet	Equivalent depth of run-off inches	Accumulated run-off, inches
955–965	147,000	147,000	0.94	0.94
965–975	174,000	321,000	1.11	2.05
975–985	202,000	523,000	1.28	3.33
985–995	234,000	757,000	1.49	4.82
995–1,005	267,000	1,024,000	1.70	6.52
1,005–1,010	147,000	1,171,000	.94	7.46
1,010–1,015	157,000	1,328,000	1.00	8.46
1,015–1,020	165,000	1,493,000	1.05	9.51
1,020–1,025	176,000	1,669,000	1.12	10.63
1,025–1,030	187,000	1,856,000	1.19	11.82
1,030–1,034	157,000	2,013,000	1.00	12.82
1,034–1,040	249,000	2,262,000	1.58	14.40
1,040–1,045	222,000	2,484,000	1.41	15.81



AREA NO.	DRAINAGE AREA (SQ MILES)		INCHES OF RAINFALL		RUN-OFF			
					INTERMEDIATE AREA		CUMULATIVE AREA	
	INTERMEDIATE AREA	CUMULATIVE	INTERMEDIATE AREA	CUMULATIVE	CFS PER SQ MILE	DEPTH INCHES	CFS PER SQ MILE	DEPTH INCHES
1	304		48.7		1.80	24.4		
2	381	685	49.3	49.0	1.77	24.0	1.78	24.1
3	536		46.0		1.39	18.8		
4	604	1,140	48.3	47.2	1.54	20.9	1.47	19.9
5	360	1,500	47.0	47.2	1.39	18.8	1.45	19.7
6	775	2,960	50.6	48.5	1.54	20.9	1.55	21.0

FIGURE 16.—*Distribution of run-off—12-year period, Oct. 1920–Sept. 1932.*

(Recent revision estimates total drainage area to be 2912 sq. miles.)

The volume between elevations 1,045 and 1,052, the maximum water level assumed in designing for the safety of the structure, is 332,000 acre-feet. The total storage from elevation 955 to elevation 1,034, the top of the spillway gates, is slightly more than 2,000,000 acre-feet.

As previously indicated, the probability of repeated floods and the necessity for prolonged storage decreases as the flood season advances. This allows a gradual filling of the reservoir, the feasibility of which was investigated by applying the record of the two largest floods for which data are available to two trial filling rates. The flood of April 3, 1886, supplied a valuable record for this study as it was an example of what might be required at the end of the flood season. This flood was the third in order of magnitude at Chattanooga and the greatest flood recorded on the Clinch River at Clinton. The other flood studied

was that of 1917 which reached a crest at Chattanooga on March 7. The more conservative of the trial filling rates permitted filling from elevation 955 on December 15 to elevation 995 on March 1 and elevation 1,005 on April 15. Four methods of operation were considered: first, that the entire flood flow would be retained until the water level rose to the top of the spillway gates; second, that a release of 10,000 cubic feet per second would be permitted after the crest at Chattanooga had passed; third, that a release of 30,000 cubic feet per second would be permitted after the crest at Chattanooga had passed; and fourth, that a free flow would be allowed over the spillway with all gates open. This study indicated that the reservoir level should not be allowed to rise above certain predetermined elevations during the flood season except when used specifically during a flood period for storing floodwater.

The filling of the reservoir which can be depended upon is limited by the stream flow. Table 6, showing run-off for the 40-year period, 1898–1937, inclusive, supplies data on the minimum flow of Clinch River. The run-off for the 6 months' period, December to May, inclusive, in the seven driest of such periods is shown in table 8.

TABLE 8.—*Clinch River at Clinton, run-off in inches for selected periods*

Year	December [1]	January	February	March	April	May	Total
SEVEN DRIEST YEARS							
1898	0.50	2.68	0.71	1.06	2.40	1.06	8.41
1900	.37	1.16	2.49	3.34	1.07	.33	8.76
1904	.63	.97	1.05	2.90	.98	1.12	7.65
1906	1.53	2.44	1.10	1.84	1.95	1.16	10.02
1910	.38	1.97	1.75	1.56	1.17	2.14	8.97
1930	1.61	1.40	2.24	2.51	1.22	1.19	10.17
1931	.61	.93	1.10	1.19	3.18	1.52	8.53
WET YEAR							
1917	1.80	5.56	3.87	10.08	2.08	.81	24.20
Average year	1.82	3.03	2.86	3.64	2.49	1.74	15.58

[1] December of the preceding year.

The lowest known flows on the Tennessee River were recorded for the months of September and October 1925. This was a very dry year, but there was a substantial run-off in the months of January and February so that the record for 1925 does not appear in the preceding table.

The study described above considered only a single flood in each case and was restricted to requirements at Chattanooga. While this study was appropriate for the purpose of showing the requirements at the end of the flood season, it was not adequate for showing what might be needed throughout the flood season. An additional study was, therefore, made to ascertain the results which might have been obtained if the reservoir had been in operation for the past 50 years, and operated for the benefit of the Clinch River, for Chattanooga and other points on the Tennessee River, and for the reduction of floods on the Ohio and Mississippi Rivers. For this purpose it was assumed that the reservoir would have been depleted to elevation 955 on December 15, the beginning of the flood season. The assumed filling was on a uniform rate with respect to volume from elevation

955 on December 15 to elevation 1,005 on April 15. The study considered the stream flow throughout the season in which operation for flood control appeared necessary.

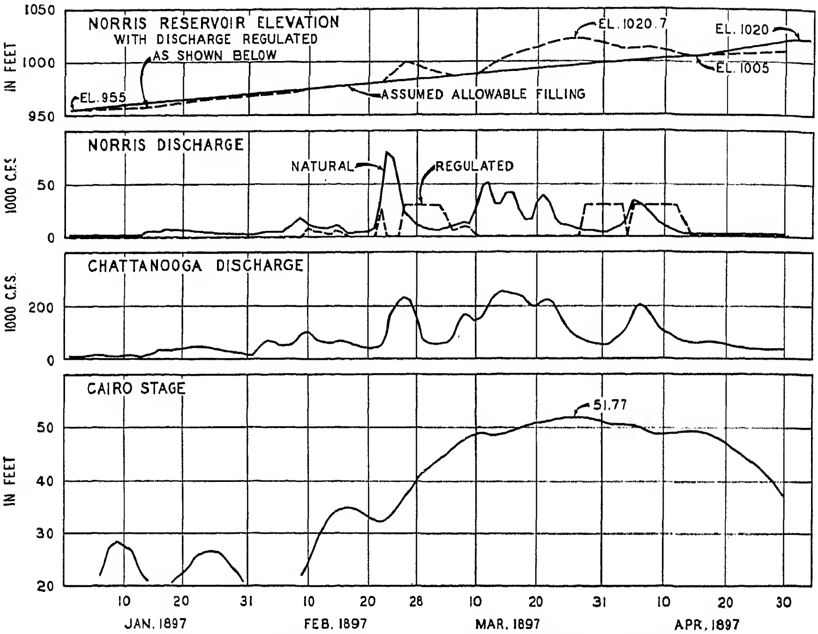


FIGURE 17.—*Possible reservoir operation during the 1897 flood.*

In the computations, it was assumed that storage would be required for one or more of the following purposes:

1. For the benefit of Clinch River below the dam to limit the maximum outflow to 30,000 cubic feet per second.
2. For the benefit of Chattanooga, to eliminate all contribution from the Clinch River above the dam during the period when the river stage exceeded 30 feet.
3. For the benefit of the Ohio and Mississippi Rivers, to withhold the entire contribution from the Clinch River above the dam during the period when the stage at Cairo exceeded 50 feet.

It was assumed that Tennessee River stages at Chattanooga could have been predicted 2 days in advance and stages at Cairo, 8 days in advance.

As soon as possible, consistent with the requirements stated, the stored water would be released at a rate not exceeding 30,000 cubic feet per second until the reservoir level was lowered to the filling curve.

Prior to construction, the longest continuous need for flood control was during the flood of March and April 1922 when, if the dam had been built, floodwater would have been retained for a period

of 45 days; the maximum water level which would have been reached would have been at elevation 1,018.9. There were periods of heavier rainfall in other years, but they occurred earlier in the season when more flood storage space would have been available. The maximum water level reached in the floods studied was in the case of the 1886 flood, which case was used in considering the permissible reservoir filling.

In January and February 1937, after the dam had been completed, the demand for flood control continued for a longer period than in the case of any of the previous floods studied. During the 1937 flood period it would have been necessary, following normal operation, to retain floodwater for 47 days and the maximum water level reached would have been slightly above elevation 1,015.

When the dam was first completed there was a general desire to have the reservoir filled. It was important to test the success of methods used in sealing and grouting the cavities and possible underground channels which might permit the seepage of water. Such a test was possible only by filling the reservoir during the period when sufficient stream flow was available. The rainfall over the Clinch watershed in January 1937 caused a stream flow substantially greater than the normal. This, combined with the general conditions accompanying the unprecedented Ohio River flood, raised the reservoir level more rapidly than had been anticipated. The spillway gates were closed to reduce flood heights at Chattanooga, at points on the river where construction work was in progress, and on the Ohio and Mississippi Rivers. Instead of a maximum of 1,015.2, the water reached an elevation a little above 1,031 or about 3 feet below the top of the spillway gates. From the standpoint of a test of the structure, very satisfactory results were found. There was no significant leakage at or in the vicinity of the dam, although there was a small amount of outflow at one point beyond the rim of the basin. However, this was not sufficient to be serious, and the property affected was acquired.

Table 9.—*Clinch River at Norris natural flow by weeks 1921 to 1935*

Mean weekly discharge in cubic feet per second

Week ending—	Week	1921	1922	1923	1924	1925	1926	1927	1928
Jan. 7		2,990	3,680	4,640	23,700	14,600	2,120	8,900	6,240
Jan. 14		4,270	7,700	4,320	10,700	16,000	2,610	3,500	2,560
Jan. 21		8,700	9,600	4,250	11,900	7,900	5,710	3,220	3,690
Jan. 28		4,580	17,300	14,600	4,780	3,730	12,900	5,430	6,810
Feb. 4		6,670	3,640	27,200	3,530	5,050	6,210	11,700	3,630
Feb. 11		8,650	2,680	24,600	3,120	4,310	8,700	7,430	7,900
Feb. 18		11,700	10,600	19,100	2,420	14,400	8,700	8,540	4,860
Feb. 25		10,500	18,700	6,760	16,100	9,000	10,600	24,500	2,750
Mar. 4		5,880	18,600	3,720	8,800	4,510	6,270	12,500	2,210
Mar. 11	10	3,740	16,600	16,000	6,330	3,200	6,220	12,700	3,540
Mar. 18	11	3,170	24,700	14,800	3,980	2,220	5,260	10,400	6,970
Mar. 25	12	3,440	8,600	15,600	6,900	4,080	4,270	5,260	10,000
Apr. 1	13	5,120	5,330	10,000	8,650	3,370	4,710	3,210	10,200
Apr. 8	14	4,330	9,200	4,130	6,110	2,520	3,020	6,210	7,240
Apr. 15	15	2,430	5,560	4,850	3,610	2,540	7,330	8,300	8,400
Apr. 22	16	10,600	5,190	7,900	8,400	2,200	6,060	9,200	11,900
Apr. 29	17	4.240	7,500	3,110	4,330	3,150	3,190	11,800	14,200
May 6	18	3,800	15,400	6,150	6,900	4,700	2,300	4,110	11,900
May 13	19	5,750	12,300	3,790	4,040	2,370	1,890	4,790	4,340
May 20	20	3,180	4,950	9,350	6,750	2,800	5,780	3,210	2,890
May 27	21	1,850	4,630	6,960	4,410	1,960	3,040	2,360	7,000
June 3	22	2,610	2,370	4,340	16,200	1,270	1,580	15,600	7,700
June 10	23	1,380	2,840	2,580	4,520	960	1,780	9,300	9,500
June 17	24	1,500	4,730	9,700	9,300	800	1,550	5,280	5,710
June 24	25	1,590	2,240	3,590	3,780	860	1,640	3,760	3,390
July 1	26	1,790	1,280	4,530	4,490	3,200	1,100	2,450	18,200
July 8	27	980	3,830	2,270	3,080	1,310	1,090	2,140	10,370
July 15	28	1,560	2,900	2,200	4,300	1,560	1,220	1,750	5,960
July 22	29	5,700	1,930	1,610	3,210	900	680	1,270	3,470
July 29	30	3,320	1,660	1,200	2,280	620	1,020	1,220	2,310
Aug. 5	31	2,060	1,420	1,620	1,490	490	1,240	1,000	1,620
Aug. 12	32	3,640	1,000	1,810	1,340	470	800	1,300	1,640
Aug. 19	33	5,420	860	3,150	1,330	480	835	2,910	3,530
Aug. 26	34	6,160	910	1,200	1,180	530	2,290	2,570	4,360
Sept. 2	35	2,120	1,100	885	1,120	400	3,140	1,140	2,470
Sept. 9	36	1,340	840	1,060	920	390	2,150	990	9,600
Sept. 16	37	1,530	780	1,730	930	390	1,120	1,320	3,320
Sept. 23	38	1,010	600	950	2,610	490	730	980	2,840
Sept. 30	39	1,060	480	1,280	3,670	460	620	680	2,380
Oct. 7	40	1,260	480	640	5,570	420	890	590	2,110
Oct. 14	41	840	550	570	1,490	440	760	610	1,930
Oct. 21	42	720	790	510	990	3,030	1,020	640	2,000
Oct. 28	43	600	560	550	790	6,540	1.420	550	3,590
Nov. 4	44	1,210	540	715	950	2,710	1,740	530	1,960
Nov. 11	45	1,810	510	820	810	6,430	1,920	580	1,760
Nov. 18	46	1,340	550	940	870	8,950	6,450	1,220	1,770
Nov. 25	47	2,800	690	700	1,820	3,470	5,420	2,040	17,000
Dec. 2	48	9,900	550	970	1,900	4,100	7,410	990	4,010
Dec. 9	49	7,600	1,760	3,280	6,960	2,660	3,570	3,170	3,840
Dec. 16	50	2,680	4,770	3,600	14,700	1,730	13,800	4,010	2,260
Dec. 23	51	2,440	14,700	3,680	2,480	2,540	14,400	8,200	3,630
Dec. 31	52	11,400	2,340	6,760	3,500	3,080	46,400	3,500	2,450
Maximum daily		26,700	38,200	57,100	37,000	30,800	60,000	41,800	45,200
Minimum daily		597	449	406	649	344	571	498	1,460
Average daily		3,950	5,240	5,370	5,080	3,390	4,860	4,860	5,480

See note at end of table, p. 43.

TABLE 9.—*Clinch River at Norris natural flow by weeks 1921 to 1935*—Continued

Mean weekly discharge in cubic feet per second

Week ending—	Week	1929	1930	1931	1932	1933	1934	1935
Jan. 7		4,850	5,760	2,170	7,010	12,200	4,780	4,920
Jan. 14		7,040	3,590	3,840	6,250	9,500	7,780	7,420
Jan. 21		10,700	2,350	2,360	3,700	4,400	2,000	5,890
Jan. 28		17,200	2,790	1,670	3,840	10,900	1,320	12,100
Feb. 4		6,140	3,100	1,090	31,600	8,700	1,020	3,760
Feb. 11		6,300	9,350	1,290	20,600	10,100	916	2,930
Feb. 18		4,190	7,660	4,510	14,300	16,600	810	10,500
Feb. 25		4,900	4,550	4,850	9,400	22,200	837	4,670
Mar. 4		25,400	4,000	2,630	4,550	5,440	19,100	10,600
Mar. 11	10	20,400	7,340	1,760	8,000	3,240	21,200	5,150
Mar. 18	11	8,100	6,350	1,660	3,650	6,240	5,800	19,700
Mar. 25	12	26,000	9,300	2,110	6,090	20,000	15,100	16,200
Apr. 1	13	14,300	3,870	7,830	12,600	5,510	20,400	25,800
Apr. 8	14	5,090	4,300	10,000	8,130	3,750	5,120	23,400
Apr. 15	15	3,520	4,400	8,200	6,890	3,720	6,690	14,000
Apr. 22	16	2,890	2,500	4,630	4,250	7,360	7,070	5,660
Apr. 29	17	4,170	1,960	11,100	6,770	3,960	5,290	6,110
May 6	18	15,500	1,570	4,220	7,600	2,730	2,560	3,360
May 13	19	16,000	1,720	4,060	4,890	7,440	1,940	5,550
May 20	20	12,000	3,750	4,350	5,550	7,300	1,700	12,100
May 27	21	20,700	5,670	3,490	2,790	4,520	1,280	8,140
June 3	22	5,290	1,610	2,990	1,790	3,550	988	4,250
June 10	23	4,380	1,160	2,010	1,280	1,690	991	4,240
June 17	24	3,460	990	1,610	1,870	2,340	1,500	2,280
June 24	25	2,420	950	1,340	2,660	1,200	1,080	3,650
July 1	26	7,420	810	1,070	4,370	1,060	900	3,100
July 8	27	6,490	670	1,180	5,230	870	1,020	3,150
July 15	28	3,390	670	2,000	2,780	1,160	1,030	2,680
July 22	29	4,080	880	1,730	1,350	830	1,830	1,700
July 29	30	2,170	840	8,200	1,020	1,370	851	2,090
Aug. 5	31	2,080	650	2,360	1,020	2,880	1,320	1,880
Aug. 12	32	1,390	500	2,700	1,010	1,630	1,480	1,390
Aug. 19	33	1,300	600	1,750	910	970	2,420	1,690
Aug. 26	34	950	1,060	4,740	890	730	2,700	1,170
Sept. 2	35	990	760	1,990	540	740	2,740	580
Sept. 9	36	810	450	1,700	500	850	1,070	890
Sept. 16	37	1,290	880	900	420	630	796	690
Sept. 23	38	1,180	600	690	400	600	614	510
Sept. 30	39	1,210	500	650	650	440	626	501
Oct. 7	40	1,480	400	570	610	360	1,480	390
Oct. 14	41	1,130	490	460	710	344	2,010	460
Oct. 21	42	780	390	420	2,410	417	1,180	440
Oct. 28	43	3,730	410	430	1,250	422	981	480
Nov. 4	44	2,580	490	650	2,100	415	1,530	730
Nov. 11	45	6,340	430	510	1,890	547	2,090	690
Nov. 18	46	13,000	1,250	450	1,420	405	1,270	5,470
Nov. 25	47	13,200	1,400	440	4,110	381	966	2,680
Dec. 2	48	3,530	680	820	1,720	388	1,830	2,170
Dec. 9	49	2,630	1,800	1,920	1,110	914	3,450	1,380
Dec. 16	50	3,260	1,380	8,600	4,470	830	1,450	2,290
Dec. 23	51	5,610	950	5,240	3,740	1,210	1,340	2,400
Dec. 31	52	5,140	2,290	6,270	18,000	2,980	2,850	1,150
Maximum daily		56,000	15,900	17,200	50,500	34,900	38,400	41,900
Minimum daily		723	371	419	359	341	566	309
Average daily		6,690	2,360	2,974	4,856	4,018	3,440	5,029

Drainage areas.—Clinton, 3,056 square miles; Coal Creek, 2,921 square miles; Norris Dam, 2,912 square miles.

NOTE.—Discharges at Norris estimated from gaging-station records at Clinton and Coal Creek, Tenn., in Bull. 40 and water-supply papers, by using proportional drainage areas.

Discharges at Norris prior to twenty-seventh week of 1927 obtained from gaging-station records at Clinton; subsequent discharges obtained from gaging-station records at Coal Creek. Discharges after the fortieth week 1934 have been corrected for storage at Norris Dam.

Navigation.

As stated in the previous discussion on the multi-purpose uses of the reservoir, the Norris project will not result in any considerable traffic on the Clinch and Powell Rivers themselves except for pleasure crafts and perhaps some for coal. The existence of a navigable pool extending substantial distances up the Clinch and Powell Rivers together with the provision for the future installation of a coal chute which is described in chapter 3 avoids the foreclosing of the development of this traffic. As a storage development, however, the project

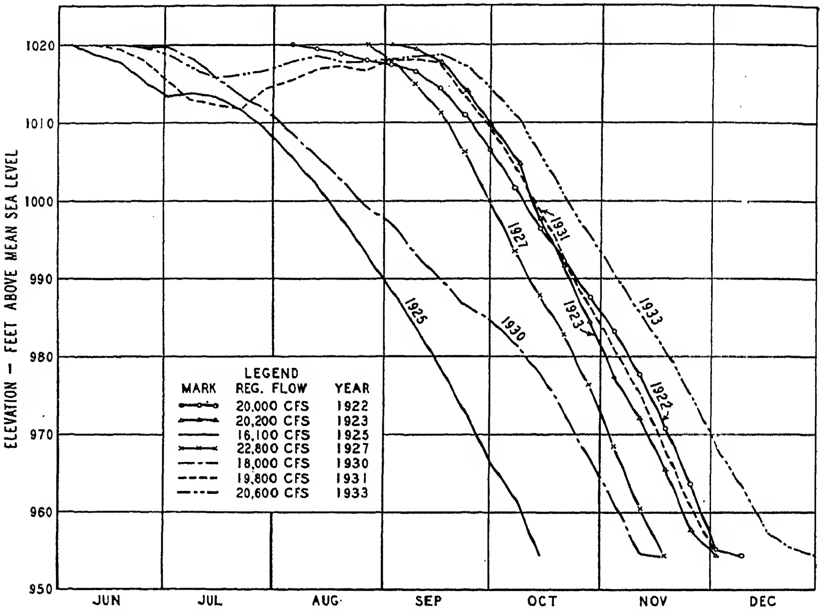


FIGURE 18.—*Study of possible Norris Reservoir operations for regulation of the Tennessee River at Florence, Ala.*

will contribute to the improvement of navigation on the Tennessee and Mississippi Rivers by means of storage releases during critical low water periods. Figure 18 shows the discharges required from Norris for seven typical years to maintain certain regulated flows of the Tennessee River at Florence, Ala.

Power.

The large reservoir that is formed for river regulation has also important potentialities for power. Only a small amount of power will be required from Norris during that part of the year when water is being stored. Water supply studies of unified operation show that during such periods the uncontrolled flow, reaching Wheeler and Wilson Dams, is sufficient for the generation of the firm system output.

In fact, on the average, the Norris Reservoir can store water about 60 percent of the time in each calendar year. However, during periods of low flow in the Tennessee River, the storage releases will generate power at the Norris plant and, in addition, will increase the available power at each down-river hydroelectric plant.

Wilson, or any other individual plant, ceases to have significance as a producer of "primary" or "dependable continuous" power, once that plant becomes part of an integrated water control system.

The net continuous power output of the Wilson plant alone is estimated to be about 39,000 kilowatts. The addition of power by the Norris plant and reservoir to a series of down-river plants is shown in table 10.

TABLE 10.—*Increase in system output due to the Norris plant*

System of main river plants	Net continuous power, kilowatts		
	Without Norris	With Norris	System increase
Wilson	39,000	137,000	98,000
Plus Wheeler	81,000	190,000	101,000
Plus Pickwick Landing	118,000	244,000	126,000
Plus Guntersville	141,000	282,000	141,000
Plus Chickamauga	183,000	330,000	147,000

All figures are net continuous kilowatts after deducting water losses and utilization losses.

The capacity required at Norris is determined by these low water releases, as well as the economic use of the storage. In the three-plant system, that is, Norris, Wheeler, and Wilson, an installation of 150,000 kilowatts is necessary to utilize the available energy at a probable plant load factor. However, the effect of storage developments on other tributaries tends to increase the length of the periods over which the storage is used during critical dry periods and thereby reduces the maximum output. With the expected addition of the Hiwassee plant to the system, as well as at least one other tributary project, it was decided to limit the capacity of Norris Dam to 100,000 kilowatts.

Under the multi-purpose operations, the Norris power output varies from 225 to 500 million kilowatt-hours annually, depending on the type of water year and the system of plants of which it is a part.

PRELIMINARY DESIGNS AND PROJECT LAY-OUTS

United States Army Engineers' design.

After a study[8] of the Tennessee River and tributaries, the United States Army Engineers recommended a dam at the present location of Norris. Their design for this dam as stated previously provided for a storage reservoir with normal pool at elevation 1,050 and with flood surcharge at elevation 1,057; a gate-controlled diversion type of spillway; a three-unit powerhouse with generating capacity of 165,000 kilowatts; and a possible barge lift consisting of a traveling crane and

[8] H. Doc. No. 328. 71st Cong., 2d sess.

tank. The tanks for this barge lift[9] were approximately 35 by 160 feet and the maximum lift was 220 feet. No barge lifts have ever been constructed in this country, but there are such installations in Germany, the highest lift being approximately 50 feet. No discharge conduits were provided, all water being discharged through the powerhouse penstocks or over the spillway. Protection was recommended at two points on the divide between the Clinch and Holston watersheds to prevent leakage from the reservoir. A dike of considerable length was designed for one point and the other low point was to be grouted to prevent underground seepage and supplemented with some riprap to protect the overburden from wave action.

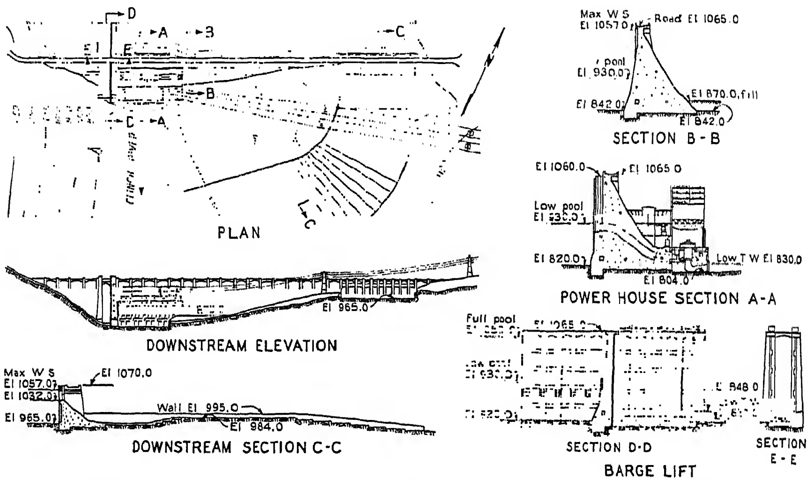


FIGURE 19.—*United States Army Engineers' design for Cove Creek Dam*

The Clinch River has been regarded as a navigable stream and with gage readings of 4.5 and 5.5 feet at Clinton would accommodate drafts of 1 and 2 feet, respectively. With a gage reading of 6 feet it is probable that a 2½-foot draft would be possible. From a study of the gage records over the period 1903 to 1933, it was shown that barges with drafts of 2 to 2½ feet could be operated about 50 percent of the time.

Extensive coal beds located along the northern border of the reservoir would provide the major portion of the potential water traffic. On the basis of rail freight movement of 1926, a total of nearly 2,000,000 tons of coal could have been transported by water had the Clinch River been developed for navigation. About 1,200,000 tons of this traffic would have originated in the area above Norris Dam.

However, if provisions were made to accommodate this potential traffic, it would have necessitated the improvement of the Clinch River below Norris Dam by the construction of three low-head dams.

[9] A barge lift was included in the preliminary studies of the U. S. Army Engineers, but was never specifically recommended by them. See Pillsbury, G. B., The Barge Lift at Norris Dam, Engineering News-Record, 113: 308–309, September 6, 1934.

In the United States Army Engineers' program this would not have been undertaken for a number of years, and then only after the demand for navigation on the Clinch warranted it. Another point which had to be considered was the construction of terminal facilities which would have to precede commercial navigation. The majority of these terminals would have been located in the reservoir area, since this is the origin of most of the potential traffic, and the variation of water levels in the reservoir would have made the cost of such facilities so high that it is doubtful if commercial interests would have found sufficient justification to provide them. Consequently, it did not seem that the large expense of constructing a barge lift—estimated by the Army Engineers to cost $3,000,000—would be justified in view of the fact that there was little, if any, navigation on either the Clinch or Powell Rivers, and the prospect of future navigation was doubtful. After a review of the Authority's studies by the board of consultants, the barge lift was omitted from later designs.

Immediately after the passage of the TVA Act, a consultant was retained by the United States Army Engineers to study the proposed design of Cove Creek Dam and to recommend a design which would more nearly fit the purposes as set up in the act in connection with the general development of the Tennessee River drainage basin for flood control, navigation, and power. It was concluded from this study that the upper elevation of the power pool be lowered to elevation 1,030 from elevation 1,050. This would allow the space between elevations 1,030 and 1,060 to be reserved for the detention of flood peaks. In the earlier design only the space between elevations 1,050 and 1,057 was reserved for this purpose. A draw-down to elevation 970 was recommended instead of the draw-down to elevation 930 as previously proposed. Aside from the above changes the consultant's design was substantially the same as the original Army Engineers' design.

Consideration of types of structures.

The United States Bureau of Reclamation was asked to study the United States Army Engineers' preliminary designs and to make revisions or alternate recommendations based on their previous experience and practices. In the preliminary designs and estimates of the Bureau of Reclamation, four types of dams were investigated—a concrete straight gravity, a concrete round-head buttress, a rolled-earth fill, and a rock-filled type. Two estimates were made on the latter type, one with a side channel spillway near the right abutment and the other with a straight channel spillway through the ridge to the east of the left abutment.

For the purposes of these alternate studies, the physical requirements were originally taken as follows:

Reservoir capacity	3,740,000 acre-feet.
Maximum reservoir level	Elevation 1,064.
Top of roadway—Concrete types	Elevation 1,070.
Top of roadway—Fill types	Elevation 1,072.
Spillway capacity	175,000 cubic feet per second.
Diversion capacity during construction	60, 000 cubic feet per second.
Outlet capacity at reservoir elevation 930	12, 000 cubic feet per second.
Penstock capacity at reservoir elevation 970	12, 000 cubic feet per second.
Number of power units	3
Rated full-load capacity of generators at 200-foot average head above tail water elevation 821.	180, 000 kilowatts.

Straight concrete gravity.—This type, shown in figure 20, consisted of a spillway in the river channel section flanked by abutment sections on either side. The spillway section was designed with three openings with provision for structural steel drum gates. The toe of the spillway extended downstream to form a concrete apron, and a sloping sill was provided at the end of the apron in order to insure the formation of a hydraulic jump and to prevent undesirable erosion below the spillway.

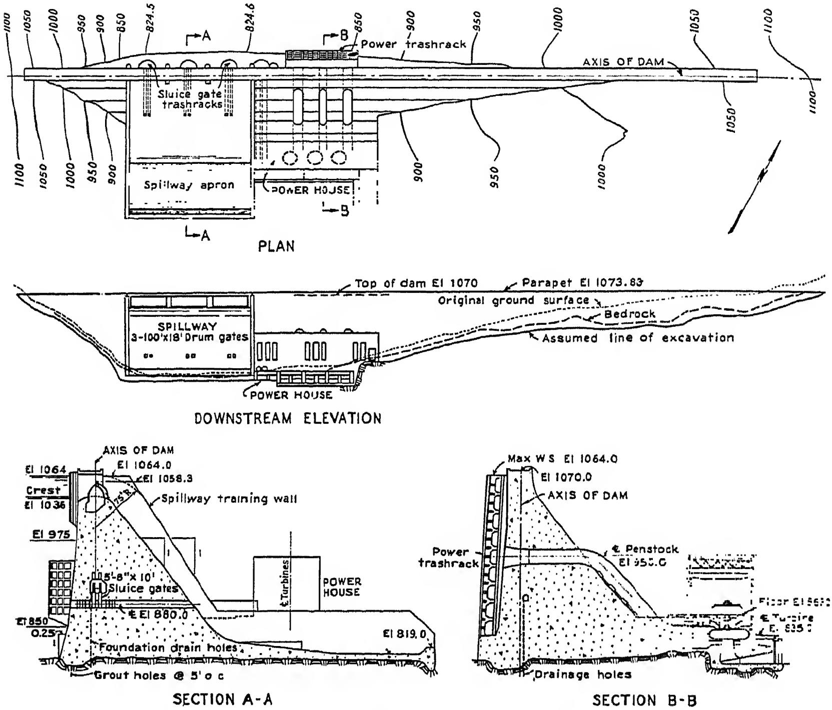


FIGURE 20.—*Preliminary design—Straight concrete gravity type dam.*

In order to regulate flow for flood control, six rectangular conduits, each controlled by two slide gates arranged in tandem, were provided in the spillway section. Two additional 102-inch diameter lined conduits were included immediately east of the spillway section for closer regulation.

The powerhouse for this design was a reinforced concrete, structural steel frame structure. The three units were served by lined penstocks 20 feet in diameter. Regulation of the flow through the penstocks was controlled with 240-inch diameter butterfly gates located in the powerhouse immediately upstream from turbine scroll cases. Provision was made for three 60,000-kilowatt units and two

1,000-kilovolt-ampere station service units. Trashrack structures were provided at the entrance of all conduits.

This preliminary design and estimate was based on a unit weight of concrete of 150 pounds per cubic foot and on an uplift assumption of full hydrostatic pressure at the upstream face varying to zero or tail water pressure at the downstream toe and applied over two-thirds of the total area. The sliding factors amounted to 0.64 and 0.61 at the base of the spillway section and at the base of the maximum abutment section respectively.

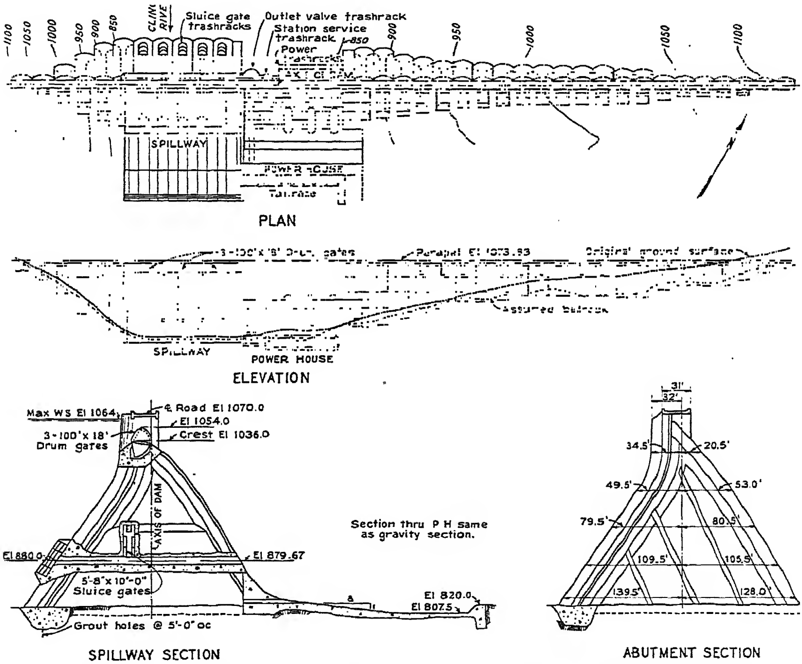


FIGURE 21.—*Preliminary design—Concrete round-head buttress type dam.*

Round-head buttress.—This design, shown in figure 21, proposed the use of the round-head buttress for the entire length of the dam except for a gravity section 280 feet in length adjacent to the east side of the spillway. This was made necessary for practical and economical construction of the outlet works and power plant. The outlet works and power plant for this design were similar to those proposed in the design of the straight concrete gravity type dam.

The spillway was located in the river section with crest elevation, length of opening, gates, and bridge the same as in the gravity type design. Drum gate chambers and bridge piers were supported by the buttresses. The downstream slope of the buttresses was protected

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by means of a reinforced concrete slab and a mass concrete apron. The latter was similar in design to the apron for the gravity type dam.

Six rectangular conduits for flood control were formed through each of the middle buttresses of the spillway section. Control was by means of slide gates.

Rolled earth fill.—This design, shown in figure 22, provided for a rolled earth fill dam with a side spillway. As a protection against wave action, the upstream face was to be protected by riprap and the downstream face was to be protected with a rock fill. A concrete core wall was provided for added safety.

A straight channel spillway was located through the ridge about 1,000 feet east of the dam. Excavation from the spillway section was equal to about 90 percent of the fill required for the dam, it being necessary to borrow the remaining 10 percent. The spillway gates were practically the same as those used in the design of the concrete types of dams.

The power plant and outlet works were located between the east abutment and the spillway—the approach channel and the trashrack and gate structures being common to both these features. The powerhouse for this design was practically the same as that planned for the concrete gravity dam.

Rock fill.—The rock fill type consisted of a well-graded quarry run rock embankment with protective facing on both slopes. The two designs of this type dam differ only as to location and type of spillway, one having a straight channel spillway similar to the spillway in the earth fill design, and the other having a side channel spillway near the right abutment of the dam. Crests of both spillways are similar with the exception that the side spillway design includes a 200-foot span concrete arch bridge along the extended axis of the dam.

Excavated material from the side channel type spillway would be suitable for the fill, while material excavated from the straight channel type would not be suitable. This fact is reflected in the cost estimates for the two dams.

Comparative cost estimates.

Table 11 shows comparative costs for the various types of structures considered.

TABLE 11.—*Comparative costs of types of structures considered*

Type of dam	Dam, spillway, outlet works	Power plant	Total cost
Round-head buttress	$13,674,367	$7,375,809	$21,050,176
Straight gravity	13,802,135	7,375,809	21,177,944
Rolled earth fill	11,941,364	13,211,086	25,152,450
Rock fill:			
Side channel spillway	15,809,511	13,211,086	29,020,597
Straight channel spillway	16,634,436	13,211,086	29,845,522

It will be noted that if the power plant is not considered, the estimate for the earth fill type is the lowest, but with power included the round-head buttress type is the cheapest. However, the difference

between the two concrete types is so small that there is little choice in regard to estimated costs. The wide variance in the cost of the powerhouses between the concrete and fill type dams is due to the necessity of longer heavy plate steel penstocks in the latter design.

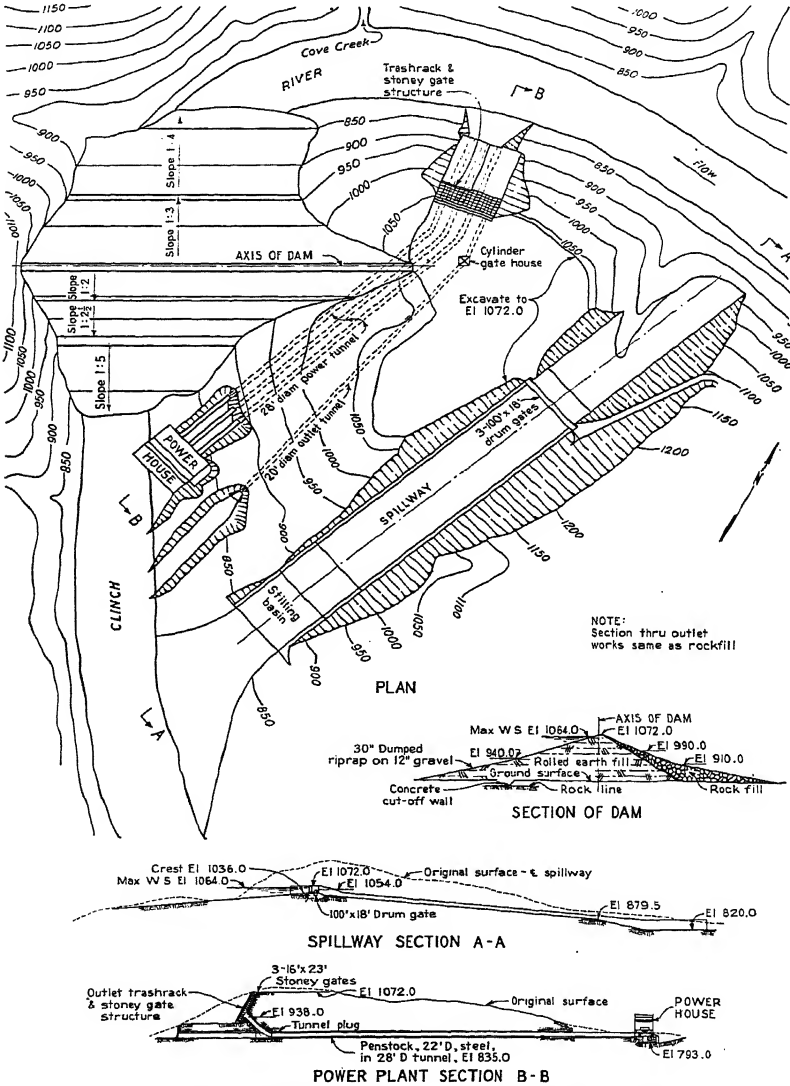

FIGURE 22.—*Preliminary design—Rolled earth fill type dam.*

From these preliminary designs and estimates, the rock and earth fill types were eliminated because of their much higher cost. With a few changes in the sections, crests, penstocks, elevations, and the use of

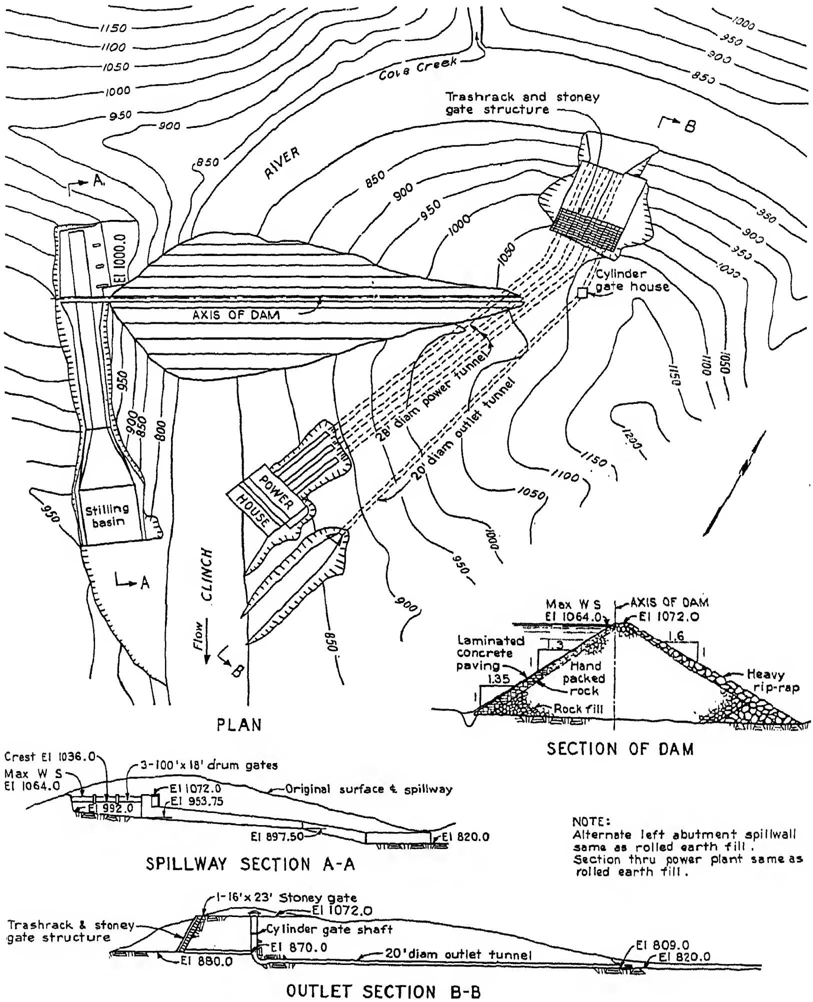

FIGURE 23.—*Preliminary design—Rock fill type dam.*

a rolled earth embankment with a concrete core wall for about 600 feet off the east end of the left abutment section, it was found that the total cost of the concrete gravity dam would be considerably lower than that of the round-head buttress type. On this basis, it was decided

that a straight gravity type would be constructed. Table 12 affords a comparison of actual TVA costs of the completed project as compared to the United States Army Engineers' and Bureau of Reclamation estimates.

REQUIREMENTS FOR FINAL DESIGN

Subsequent to the detailed investigations, studies, and estimates, the general requirements for the final design were determined and may be summarized as follows:

Location	Near Cove Creek at site A.
Type	Straight concrete gravity.
Top of roadway	Elevation 1061.
Maximum high water (structural design)	Elevation 1052.
Probable high water	Elevation 1047.
Top of spillway gates	Elevation 1034.
Spillway crest	Elevation 1020.
Bottom of outlet conduits (maximum possible draw-down).	Elevation 860.
Capacity of spillway	200,000 cubic feet per second.
Capacity of outlet conduits	40,000 cubic feet per second.
Number of power units	2.
Total capacity at 0.9 power factor	100,000 kilowatts.

These requirements necessitated some reservoir treatment which, however, was not a serious item and consisted mainly of the construction of a small earth dike near the Loyston Divide to prevent leakage of reservoir water over the reservoir rim at extremely high stages. While it was known that there was a likelihood of some leakage through the reservoir rim near the dam abutments, it was not considered economical to seal these entire areas by grouting until it was known how serious this problem might be. The more notable areas of weakness for a distance of approximately 2 miles from the east abutment and for several thousand feet from the west abutment were grouted to some extent.

GEOLOGY OF THE NORRIS RESERVOIR REGION

The first geologic mapping in the present Norris Reservoir area was done by the United States Geological Survey between 1889 and 1899, as part of the Maynardville, Morristown, and Briceville folios, all of which have been published. Since much geological information of scientific value would be lost by flooding, a more detailed geologic map of the reservoir area was made upon newer and more accurate base maps. In conjunction with this work examination was made of hundreds of claims for mineral damages in the reservoir.

GEOLOGIC DESCRIPTION OF REGION

The reservoir area is in the western portion of the Valley and Ridge physiographic province, between the Cumberland Plateau and Clinch Mountain. The area may be subdivided into three physiographic divisions:

1. The faulted ridge and valley area on the southeast, consisting of parallel ridges and valleys formed on various belts of steeply dipping rocks.

TABLE 12.—*Comparison of Norris Dam cost with preliminary estimates by U. S. Bureau of Reclamation and U. S. Army Engineers*

[General expenses have been prorated to items shown]

Description	Norris Dam actual cost			U. S. Bureau of Reclamation estimate as of September 1933			U. S. Army Engineers estimate as of March 1930 [1]		
	Quantity	Unit cost	Amount	Quantity	Unit cost	Amount	Quantity	Unit cost	Amount
Improvements to station site		Lump	$209,740.15		Lump	$1,265		Lump	$123,363
Cofferdams and care of water		Lump	558,494.19		Lump	335,225		Lump	246,726
Stripping for embankment		Lump	2,509.06	6,800 cubic yards	$0.44	3,011			
Excavation and backfill	487,985 cubic yards	$2.07	1,009,986.50	661,300 cubic yards	$3.36	2,220,391	751,476 cubic yards	$2.57	1,933,124
Embankment		Lump	[2] 503,809.53	109,100 cubic yards	$0.93	101,757			
Backfill				2,000 cubic yards	$1.26	2,530			
Exploration of foundations		Lump	38,897.83						
Foundation preparation and grouting		Lump	730,310.84		Lump	353,464		Lump	173,483
Joints, stops, and foundation drains		Lump	143,473.91						
Concrete in dam	936,286 cubic yards	$7.58	7,093,199.46	926,740 cubic yards	$8.45	7,832,513	821,116 cubic yards	$13.59	11,162,926
Gates, trashracks, and other dam appurtenances		Lump	974,798.34		Lump	1,583,157		Lump	[3] 2,443,683
Subtotal			11,265,219.81			12,433,313			16,083,305
Powerhouse building		Lump	1,152,622.74		Lump	[4] 828,795		Lump	[4] 745,729
Penstock piping and tailrace paving		Lump	204,946.37						
Turbines and governors	132,000 horsepower	$6.42	847,603.78	270,000 horsepower	$5.63	[4] 1,518,968	156,000 horsepower	$6.03	[4] 941,506
Generators and exciters	100,800 kilowatts	$12.38	1,248,223.09	198,000 kilowatts	$10.51	[4] 2,081,200	165,000 kilowatts	$8.23	[4] 1,357,489
Electrical equipment	100,800 kilowatts	$13.00	[5] 1,310,350.85	198,000 kilowatts	$5.81	[4] 1,151,272	165,000 kilowatts	$7.09	[4] 1,169,481
Mechanical equipment		Lump	245,040.37		Lump	[4] 236,311		Lump	[4] 231,305
Subtotal			16,274,007.01			18,249,859			20,528,815
Reservoir (includes land cost and acquisition cost, relocation and protection costs, reservoir clearance and rim treatment, Buffalo Creek Divide dike.)		Lump	14,950,535.00			(6)			13,346,328
Access road and roadway on dam		Lump	275,900.34			(6)	Access road and railroad		1,973,808
Total			31,500,442.35						35,848,951
							Estimated interest during construction.		1,691,692
							Total shown in H. Doc. No. 328.		37,540,643

[1] Estimate includes barge lift.
[2] Includes core trench excavation and core wall concrete in addition to rolled fill.
[3] Includes barge lift equipment. Total does not afford a good basis of comparison.
[4] Estimate based on three units.
[5] Includes outdoor substation costs.
[6] No estimate.

2. Dividing ridge, or Powell River anticline, consisting of a broad arch of nearly flat-lying Knox dolomite into which Powell River and its tributaries have incised a dendritic drainage pattern, and

3. Powell Valley, which is similar to the first division but has fewer ridges.

Stratigraphy.—With the exception of one small area, the rocks of the Norris Reservoir area are all sedimentary, consisting of limestone, dolomite, shale and sandstone, and gradational varieties of these types. They are of Paleozoic age, ranging in age from early Cambrian to Devonian, as shown on the generalized stratigraphic column.

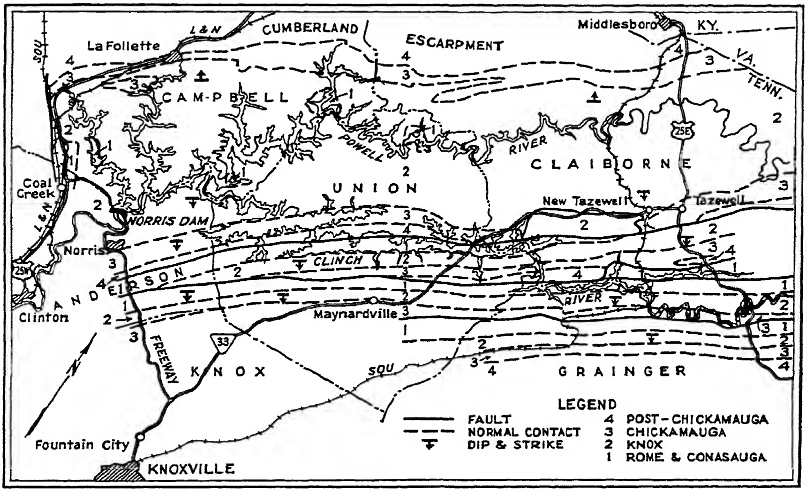

Figure 24.—Geology of the reservoir region

GEOLOGIC COLUMN FOR THE NORRIS RESERVOIR AREA

Devonian	Chattanooga shale.	
	Becraft sandstone.	
Silurian	Hancock dolomite.	
	Clinton formation.	
	Clinch sandstone.	
Ordovician	Juniata—Sequatchie formation.	
	Reedsville shale and limestone.	
	Trenton limestone.	
	Lowville—Moccasin limestone	
	Ottossee shale and limestone	Chickamauga limestone.
	Stones River limestones	
	Post—Longview limestone and dolomite	
	Longview dolomite	Knox dolomite.
	Chepultepec dolomite	
Cambrian	Copper Ridge dolomite	
	Nolichucky shale and limestone	
	Maryville limestone	Conasauga formation.
	Rogersville shale	
	Rutledge limestone	
	Rome shale, sandstone, and limestone.	

Of the several formations listed, three groups comprise the most widespread and most important rocks of the area: the Rome, the Knox, and the Chickamauga.

The Rome formation.—This is a heterogeneous formation consisting of red, green, and brown shales; thick, coarse sandstones; sandy shales; and massive beds of limestone and dolomite. Its exposed thickness here is about 1,500 feet. As a whole, it is resistant to weathering and forms the "comby" ridges which are so conspicuous throughout the area. These ridges have a steep scarp slope, and a slightly less steep dip slope, and provide a barrier to transportation routes, except where they are cut by water gaps.

It crosses the reservoir area in three separate parallel belts, making up the foundation rock at the following locations: Big Ridge Dam, the new railroad bridge at Days Mill, and the north abutment of the new Clinch River highway bridge. The new Southern Railway tunnel at Days Mill cuts through the lower part of the Rome in one of the typical ridges.

Excepting the limestones of the lower part, the Rome is an insoluble formation, and the thick sandstone beds are sufficiently strong to support structures so that the formation is well adapted for the above-mentioned structures. It has, however, one important weakness in the susceptibility to sliding on dip slopes. The interbedded shales, when moistened, afford good surfaces on which the sandstone beds slip when any supporting material is removed. This was experienced at the south portal of the Days Mill tunnel and in the Indian Creek road relocation.

Conasauga group.—This group consists of 800 to 1,700 feet of massive limestones and a thick series of calcareous shales; but in the upper (Nolichucky) portion, the shales and limestones are more intimately interbedded. Shales are more abundant in the western part of the area. The Conasauga as a whole is a soluble, nonresistant formation, and forms valleys which are usually followed by longitudinal roads. The south abutment of the new Clinch River highway bridge is on the lower bed of the Conasauga. Most of Big Ridge Reservoir is underlain by this group.

Knox dolomite.—The most important and widespread group of rocks in the reservoir area is the Knox dolomite. Usually considered as a single unit in all general discussions, it is, however, divisible into at least four distinct formations which may be recognized by careful study of their fossils and lithologic character. The total thickness is about 2,750 feet.

The lowest formation of the group is the Copper Ridge, consisting of about 1,000 feet of massive, coarsely granular dolomite. Small amounts of bituminous material present give many of the beds a dark color and a decided fetid odor when freshly broken. Upon weathering, this formation yields a deep red soil and a great amount of chert in loose fragments which litter the surface varying in size up to a few feet in diameter. Different types of chert may be recognized at several horizons but this requires detailed study. Norris Dam is built upon this formation and practically all the Powell River area of the lake is underlain by it.

The next formation—the Chepultepec dolomite—contains beds very similar in lithology to those of the Copper Ridge but less massively

bedded and usually less well-exposed. It is about 900 feet thick and yields large masses of chert which are usually more porous than those of the Copper Ridge. It is distinguished by a zone of sandy dolomite at its base. Leaching of the carbonate cement from these beds produces a porous sandstone which attains a thickness of 14 feet on Cove Creek. This sandstone is a good aquifer and many springs issue from it.

The Longview and Post Longview beds are a series of about 1,800 feet of light gray crystalline dolomite and dense blue limestone. They also yield great quantities of residual chert and, in some cases, thick beds of chert.

As a whole, the Knox dolomite is defined as a soluble unit, and any portion of it may support extensive underground drainage. Since low dips are favorable to underground drainage and since the Copper Ridge and Chepultepec occupies the largest areas of low dip, it is in these formations that the largest sinks and caves are to be found. These are in the central peninsula and west side of the Powell River and both sides of the Clinch River below its junction with the Powell.

Deep-sided ridges with shallow intermediate valleys is the usual topographic expression of the Knox group. The ridges are most commonly developed upon the Copper Ridge and Longview formations. The Post Longview occupies the dip slope of the ridge and the side of the adjacent valley of Chickamauga limestone.

Chief utilization of the Knox dolomite has been as road material and for masonry purposes. All the formations of the group contribute to these uses, which have been entirely local. Norris Dam quarry is in the Copper Ridge dolomite.

Chickamauga limestone.—The general term Chickamauga limestone embraces several different limestones of more or less similar character whose total thickness is about 1,500 feet. As a unit they are quite soluble and nonresistant to weathering so that they are found in continuous valleys, frequently occupied by longitudinal streams and generally followed by through roads. Such valleys as the Powell Valley, Big Valley, Dutch Valley, and Clinch Valley are illustrative of this.

Generally the limestones of the Chickamauga are thin bedded, though some dense blue limestones of the Stones River and coarsely crystalline limestones of the Ottossee exceed 3 or 4 feet in thickness. Both shaly limestones and rather pure limestones are present in all the formations of this group, the former being more prominent in the Lowville-Moccasin, the Ottossee, and the Trenton, and the latter being found in the Stones River and certain beds of the Ottossee. All are highly fossiliferous, and good fossil-collecting localities are abundant throughout the area.

Of special interest is the occurrence of at least two beds of metabentonite (altered volcanic ash). These attain thicknesses as great as 4 feet and are very widespread, not only in the Norris area, but throughout the eastern United States. All limestones of the group are highly soluble and both sinks and caves are of frequent occurrence. The Buffalo Divide north of Andersonville is underlain by these limestones. They have no economic uses except as road metal and as slabs for flagstones, coping, etc. For road metal, the Lowville and Stones River beds are most often used in this area.

Miscellaneous formations.—The Reedsville and the Sequatchie Juniata formations are thick shale series occurring high on the scarp slopes of the Clinton Ridges and are of no engineering or economic significance. The Clinch sandstone is present only locally. The Clinton formation which overlies it consists of about 450 feet of sandstone and sandy shales and forms such continuous deep ridges as Lone Mountain and Big Ridge. It contains a thin bed of fossiliferous iron ore, too thin to be worked profitably, except at La Follette, where it once supported a furnace.

Above the Clinton, the formations are too thin and too local in their occurrence to be of engineering or economic importance to the area. Of special interest is the occurrence on Clinch River near the gourd of two small bodies of mica-peridotite, an igneous rock which has been intruded into the sedimentary rock and is younger than the youngest rock of the area. This is the only intrusive igneous rock known to occur in Tennessee west of the Blue Ridge Province.

Structure.

Norris Reservoir area may be subdivided into two distinct structural units, the fault-block area of the southeastern portion, and the Powell River anticline occupying the northwest portion.

Fault-block area.—Originally deposited upon the sea bottom in a horizontal position, the sedimentary beds were later subjected to intense compressive forces acting from the southeast, causing them to fold and in many cases to break on the axes of these folds. Thus the portion of the reservoir area southeast of Lone Mountain and Big Ridge has been broken into three major and several minor blocks which have been thrust northwestward over the adjacent areas, so that the lower Cambrian beds now rest upon younger Ordovician and Silurian formations. The base of each block is an inclined thrust fault, usually in the lower part of the Rome. In this area the rocks are generally inclined at angles of 25° or more, their strike is north 40° east to north 60° east, and the direction of their dip is, with very few exceptions, toward the southeast. The effect of erosion upon these structures is to produce a pattern of long, narrow, parallel strips of rocks, the harder ones forming ridges and the softer and more soluble ones forming valleys. This can be observed clearly in driving from Knoxville to Norris where 5 separate fault blocks containing the same series of rocks and at least 40 different belts of rocks, all dipping southeast, can be seen. The Rome formation is crossed at Sharps Ridge, Beaver Ridge, Bullrun Ridge, Pine Ridge, and part of Lone Mountain.

Powell River anticline.—Lying between the Cumberland Plateau and the fault-block area just described is the broad arch of the Powell River anticline, so named because the Powell River flows approximately along its axis. Throughout the major portion of the anticline, the rocks are almost horizontal, or inclined at very low angles. This is the large area of Knox dolomite of the central peninsula and adjacent areas and is the area upon which Norris Dam is located. Local domes on the major structure caused the older Conasauga beds to come to the surface at Lead Mine Bend, Davis Creek, and other localities. On the southeast side of the anticline,

the Chickamauga and younger beds form flanking belts which dip southeastward at about 25°. These same beds flank the arch on the northwest side, where their dip is much steeper and often vertical as they pass beneath the Cumberland Plateau.

Although a distinct unit as far as the reservoir area is concerned, the Powell River anticline is actually only a small part of a very large thrust block which extends well beyond the limits of the Norris Reservoir area. This block moved on an essentially horizontal plane for several miles to the northwest.

SOCIAL AND ECONOMIC STUDIES

Since the Cove Creek site had been under consideration for a number of years, preliminary investigations covering the engineering and geologic aspects of the project had been made by the United States Army Engineers and others prior to 1933. However, beyond ascertaining the probable value of the land to be acquired for the completed project, no studies had been made relative to family removal, problems of access to isolated areas, or the effects of governmental purchase of real property on local government finance. Although these studies are normally a part of preliminary investigations, time was not available for such investigations prior to the starting of actual construction because of the urgent need to relieve unemployment. The investigations were started as soon as possible, however, and served as the basis for treatment of the social and economic problems.

EFFECT OF GOVERNMENT PURCHASE OF REAL PROPERTY ON LOCAL GOVERNMENT FINANCE

A study was made to determine the direct and immediate effect which the acquisition of the reservoir property and its removal from the tax base might be expected to have on governmental finances. This study was restricted to county finances, because there were no other local taxing jurisdictions within the area purchased, and the amount of State revenue involved was small.

Approximately 152,000 acres of land situated in Union, Campbell, Claiborne, Anderson, and Grainger Counties were purchased for the project. This purchased land constituted approximately 13 percent of the land area of the five counties. The population of these counties, which numbered 95,000 in 1930, was then about 97 percent rural with agriculture of a self-sufficient type the predominant occupation. Mining was also important in Anderson, Campbell, and Claiborne Counties, which together produced over 3,000,000 tons of coal in 1929. Manufacturing was negligible, except in Anderson and Campbell Counties. In 1929 the total value of products manufactured in Campbell County exceeded $1,500,000, and while no comparable figures are available for Anderson County, the number of persons gainfully employed in its manufacturing and mechanical industries was 1,153, or 18.5 percent of all workers. This number greatly exceeded the numbers for the other counties in the Norris area.

Although over 3,500 families were reported affected to some degree by the reservoir, the population of the five-county area has not been decreased to the full extent indicated. With an estimated 2,500

families remaining within the reservoir counties, the net population loss is approximately 5 percent of the 1930 total of 19,736 families.

The property purchased was valued at approximately $8,670,000. A total of 125 nontaxable tracts was appraised at $258,936 of which about $140,000 was for county property and most of the remainder for churches or other property owned by religious organizations.

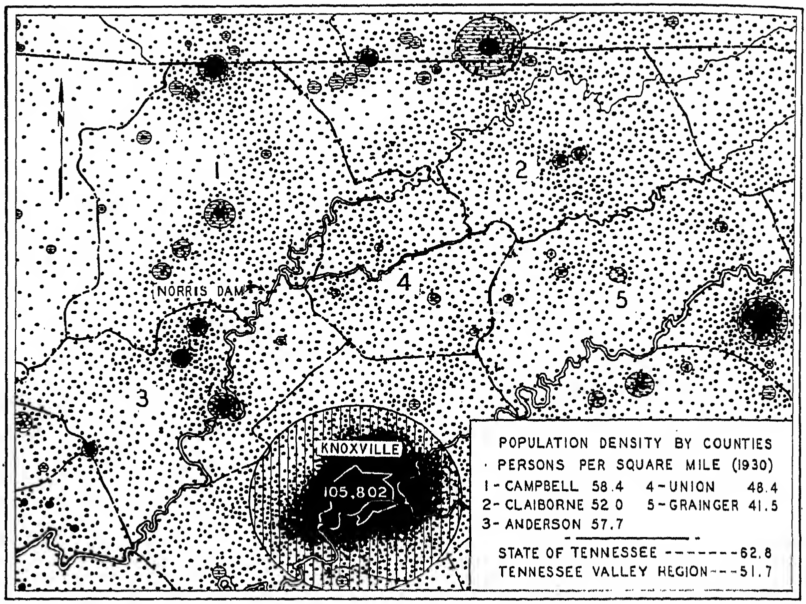

FIGURE 25.—*Population density map, 1930.*

Loss of tax base.

The percentage of loss in the property tax assessment in the purchase area is considerably smaller than the percentage loss in the acreage of taxable property since slightly over 40 percent of the assessed value for the five counties consisted of public utilities, town lots, and personal property which were affected but little by the reservoir. Campbell and Anderson Counties, which have the largest total property assessments, contained almost 88 percent of all the utility property in the five counties and the major portion of the assessed value of city and town real estate. A summary of the pertinent data on acreage, assessed valuation, and annual revenues for the reservoir counties is given in table 13.

Some personal property was removed from the tax base of the five counties through the relocation of between 1,000 and 1,100 families in other counties. However, the taxable personal property of families moving from the reservoir but relocating within their respective counties should remain on the tax rolls.

TABLE 13.—*Acreage, assessed valuation, and annual revenues for reservoir counties*

[Based on data available in 1934]

Acreage:	
Total in 5-county area	1,159,040
Acres purchased—(approximate)	152,800
Percent of 5-county area	13.2
Assessed valuation—1934:	
5-county total	$24,104,427
Purchased—amount	1,995,242
Percent of 5-county total	8.3
Annual revenues:	
5-county total—1933–34 average	$1,306,346
Purchase area—1934 tax levy	50,613
Percent of 5-county total	3.9
Total county tax base by kinds of property—1934:	
Real estate:	
Acreage percent	59.1
Lots do	11.3
Personal property do	5.8
Public utilities do	23.8

The remaining property purchased by the Authority was rural real estate with the exception of a few lots in the unincorporated villages of Loyston and Caryville and a short section of the Vasper-La Follette branch line of the Southern Railway. A sum of approximately $8,300,000 was paid to owners of taxable real estate, and about two-thirds of these landowners have continued to reside in the Norris counties. Authority payments for the land averaged approximately $55 per acre, as compared with an average assessed value of $13 per acre. The local investment of some of this purchase money has added to the physical amount of taxable property in the area and the expenditures made in the area have stimulated existing property values. Over 800 new houses have been built more or less directly as a result of family relocations in the area immediately surrounding the reservoir. The census reports a 42 percent increase in farm values for this area between 1930 and 1935. Because of an inefficient taxing system, however, assessed values have failed to increase in any way commensurate with increases in real values.

Loss of revenue.

As shown by the preceding tabulation, the percentage of loss in revenue because of Authority purchases is less than half the percentage of loss in assessed value, on the basis of 1934 figures. The purchase area constituted 8.3 percent of the tax base but would have provided only 3.9 percent of the aggregate revenues of the five counties. In 1934 State grants-in-aid and a small amount of State-collected, locally shared taxes supplied slightly more than half of the total revenue of these counties.

1933–34 Average revenue—five counties affected by Norris Reservoir

Average annual revenue—1933–34	$1,306,346
Distribution by source:	
Property tax percent	46.0
Miscellaneous county revenue do	3.8
State aid do	50.2

The two-year averages may reflect depression conditions, but the percentages should be representative since Federal aid is excluded. State

aid was much larger in 1933 than in 1934, whereas property tax collections were greater in 1934.

Relative to other Tennessee counties, State aid has declined somewhat in the Norris counties following the removal of reservoir families. The aid for education, which constitutes about half of the total, and the aid for highways, which constitutes most of the remainder, are both distributed partially on the basis of population. The reduction in State aid in the Norris counties has been relative rather than absolute, however, because State funds for distribution to counties have been increased for the State generally. There also has been some decrease in the needs for revenue in the Norris counties due to the smaller population and the smaller area to be served. On a unit basis, therefore, the amount of revenue for county services in the Norris area has been increased.

The estimated losses in tax levies due to acquisition of property by the Authority are $50,613 for the counties and $1,596 for the State. These estimates are based on 1934 valuations and 1934 tax rates. By 1937, tax rate increases had more than offset tax base reductions, and the aggregate county levies for that year were 2 percent greater than 1934.

The payment of all back taxes as a prerequisite to land purchases by the Authority provided considerable revenue to the Norris counties which had been having difficulty in making collections. Collections of current and delinquent taxes for the counties as a whole rose to a peak in 1936. By 1937 the abnormal situation created by tax clearances of TVA lands had abated, and total collections dropped accordingly.

County debt.

The acquisition of private property and its removal from the tax base left the existing county debts unmodified. The total bonded indebtedness of the five counties in the reservoir area in January 1934, at the beginning of the Authority's land purchase program, was $4,481,000. By January 1938 the counties had reduced their bonded debt to approximately $4,148,000. This reduction was made possible largely by TVA payments for county property flooded and for purchase of private property which in turn resulted in greatly increased back-tax collections. The net reduction was accomplished in the face of $185,000 of new improvement bonds issued since 1934. The ratio of bonded indebtedness to assessed value increased from 17.7 to 18.9 percent over the 4-year period.

TABLE 14.—*1934 indebtedness of the 5 Norris Reservoir counties*

Bonds:	
Road	$3, 524, 000
School	472, 000
Other	485, 000
	4, 481, 000
Less road bonds assumed by State	1, 313, 253
	3, 167, 747
Floating debt	538, 739
Total bonds and floating debt	3, 706, 486
Less sinking fund	174, 338
Net total—5-county debt	3, 532, 148

Expenditures for public property and other purposes.

The aggregate value of property owned by the counties has been modified only slightly by the acquisition of reservoir lands. A total of $134,585 was paid for 40 school sites and buildings. A poor farm was purchased from Union County for $5,501 and a rock quarry from Campbell County for $250.

Roads have been relocated to provide services approximately equivalent to those available before the existence of the dam and reservoir, and at costs considerably in excess of the original outlay for the flooded facilities. A school building has been constructed in the town of Norris, in Anderson County, at a cost of approximately $170,000. Operation of this school is by the Authority, primarily for residents of the town of Norris; also, pupils are enrolled from outside the town under a contractual arrangement with Anderson County.

Effects on Union County.

The effects of the land purchases and the removal of property from the tax base are felt more keenly in Union County than in any of the other four counties since 42 percent of the county area and approximately the same proportion of the assessed value of all taxable property in that county have been acquired.

Interruption of the financial and administrative affairs is more significant than the percentages of loss in area and assessed value indicate. In the first place, the reservoir divides the taxable area of the county into four parts accessible to each other only by roads leading through adjoining counties. This circumstance tends to increase the cost of administration relative to the population and area served. In the second place, Union County already was small in area and relatively poor; hence even a small absolute cut in revenue tended to interrupt financial operations.

It has been estimated that more than 1,100 Union County families were directly affected by reservoir land purchase, of which over 600 relocated outside the county. In terms of the 1930 population this shift in reservoir families has reduced the number of persons in the county by approximately 25 percent but has increased density per square mile from 48 to 63 for the portion of the county outside the Authority's purchase area. However, it is not known to what extent reservoir family relocations displaced other families in the county, nor to what extent other population changes have taken place since 1930. The decline in total population, however, together with a much larger proportionate reduction in the privately occupied area to be served, has decreased the amount of revenue needed to provide county services comparable with those previously rendered.

Loss of tax base.—The Authority bought more than 63,000 acres of Union County land with an assessed value of $1,087,978, which was 42.8 percent of the total real estate assessment and 41.5 percent of the total property assessment of the county. Real estate constituted 97 percent of the 1934 assessment. None of the public utility properties was affected directly by the purchases of the Authority. The entire personal property assessment in 1934 was less than 1 percent of the total property assessment, so that the reduction in the amount of personalty attributable to family relocation is negligible.

A part of the initial loss in property tax base should have been offset by increased purchasing power and property improvements as a result of payments made by the Authority to landowners who have remained within the county. The lands within the Union County purchase area cost the Authority almost $3,438,000 through June 30, 1937, which is more than three times their assessed value. However, the compensating factor has not yet operated to strengthen tax base due to a faulty tax assessment procedure. By 1937, because of assessment reductions outside the TVA purchase area, tax values in Union County had fallen 50 percent from 1934, only 43 percent of which was caused by TVA purchases.

Loss of revenue.—The immediate removal of $1,087,978 from the property tax base deprived Union County of a tax levy of $24,153 at the 1934 rate of $2.22 per $100 of assessed value. This sum was equal to 17.6 percent of the average total annual receipts of the county for the fiscal years 1933 and 1934. According to actual property tax collections during these years, the revenue loss was $23,697 annually, or 17.3 percent of the total county revenue. The property tax collections, particularly those of 1934, included substantial amounts of back taxes. On the other hand, for both 1933 and 1934 tax delinquency was generally high.

This relatively low percentage of loss in revenue is explained by the fact that a large proportion of the total revenue is derived from sources other than the property tax. The principal revenue source for the county is State aid. For the fiscal years 1933 and 1934, an annual average of 54.2 percent of all county revenue was received from State grants-in-aid and State-collected taxes.

The net loss to Union County of some 600 reservoir families, which represent nearly one-fourth of the county population, has as yet caused little or no reduction in the amount of State aid. Grants for elementary education, which for Union County were frozen at $29,468 from 1932 to 1937, were originally determined primarily on the basis of school attendance. Under the present law the county will continue to receive this amount because of a guaranteed minimum provision. The county's share of the State gasoline tax for road purposes, averaging $28,402 for 1933 and 1934, is distributed partly according to population, but family removals will not affect this before the 1940 census. The need for school and road revenue from the State is smaller since both the population and area to be served are greatly reduced. The State has continued to pay interest and principal on certain county road bonds that it has designated for reimbursement.

Nontaxable property.—School, church, cemetery, poor farm, and other nontaxable property in Union County purchased by the Authority amounted to a total of $106,324. Roads in Union County were relocated by the Authority to provide the area remaining with services comparable to those available before the reservoir was formed, and a further payment of $141,000 was made for final damages to other roads and bridges. This money has been applied to the reduction of the county road debt. It is estimated that in Union County the reservoir flooded 92 miles of highways and 1,817 feet of bridges which together originally cost $252,700. The Authority

relocated about 9 miles of county highways at a direct cost of almost $130,000. A new bridge and about 7 miles of road were constructed within the county on Tennessee Highway No. 33 at a cost of about $683,000. These sums expended by the Authority, plus the cash payment of $141,000, make an aggregate direct expenditure of approximately $954,000 for roads and bridges in the county.

Indebtedness.—Union County has reduced its bonded debt from approximately $297,000 in January 1934 to approximately $106,000 in January 1938. This reduction was made possible largely through the payment of $41,000 by the Authority for county schools within the purchase area, the $141,000 final settlement for county roads and bridges damaged by the reservoir, and back-tax clearances of TVA lands. The ratio of bonded debt to property assessment was reduced from 11.2 percent in 1934 to 8 percent in 1938.

RELOCATION OF PEOPLE LIVING IN FLOODED AREA

In order to secure information relative to all families in the flowage area, nearly 3,000 families who were required to move to new locations were interviewed during the summer of 1934. Detailed questionnaires were filled out for each family.

The purposes of the study were:

1. To secure information that would be the basis for the assistance to be given to individual families in solving their relocation problems.
2. To provide the basis for future studies which would determine the relative conditions of these same families after resettlement.
3. To gather basic data of a definite area which would be available for use in the planning activities of the Authority.

About two-thirds of the families in the reservoir area owned their homes. This large proportion of ownership, which was consistent throughout the counties affected, stands in contrast to the entire State of Tennessee in which only about one-half of the farm homes are occupied by their owners. For study purposes, the Norris families were considered all native white since less than a half-dozen Negro families and only two foreign-born heads of families were included.

Tenant and landowner families of the area were further classified according to economic status. Tenants were divided for this purpose into relief and nonrelief groups. The survey disclosed that 334 tenant families, or slightly more than one-third of the total, received relief in 1933 or 1934. The value of the land owned was used in classifying owner families.

Families of the Norris flowage area

[Based on data available in 1934]

		Percent
Total homes in flowage area	2,841	
Owners		65.6
Tenants		34.4
Landowning families—total	1,864	100.0
Appraised value:		
Under $1,000	417	22.4
$1,000 to $3,999	991	53.2
$4,000 and over	456	24.4

Families of the Norris flowage area—Continued

		Percent
Tenant families—total	977	100.0
On relief	334	34.2
Nonrelief	643	65.8

More than one-half of the landowning families had property valued between $1,000 and $3,999.

TABLE 15.—*A summary classification of families according to their probable dependence upon outside assistance (1934 data)*

	Landowner familes		Tenant familes		All famlies	
	Number	*Percent*	*Number*	*Percent*	*Number*	*Percent*
Total families	1,884	100	1,018	100	2,902	100
I. Completely independent	448	23.8	111	10.9	559	19.3
Families who have:						
Already secured location	418	22.2	104	10.2	522	18.0
Planned to retire with own support	30	1.6	7	.7	37	1.3
II. Advisory service only	1,333	70.7	431	42.3	1,764	60.8
Families who should be:						
Full-time farm owners	835	44.3	48	4.7	883	30.5
Full-time tenant farmers	40	2.1	120	11.8	160	5.5
In professional and industrial work	62	3.3	99	9.7	161	5.5
Self-supporting through agriculture-industry coordination	396	21.0	164	16.1	560	19.3
III. Both advisory and financial assistance	103	5.5	476	46.8	579	19.9
Families who may be:						
Self-supporting after a period	61	3.3	439	43.1	500	17.2
Submarginal	42	2.2	37	3.7	79	2.7

In order to secure a reliable estimate of the number and kinds of problems, some means of classifying the families was necessary. An attempt was made to scale the families according to the probable degree of their dependence upon outside aid and relocation as shown in table 15. The first of three major groups is made up of those families which would probably require neither financial nor advisory assistance. The second group is comprised of families which were at present self-supporting but would probably need or desire advice in relocating satisfactorily. The third group is made up of those families which probably could not move and re-establish themselves without both financial and advisory assistance. Of 2,902 families in the area, it was found that 559 were completely independent, 1,764 needed advisory assistance only, and 579 needed both advisory and financial assistance.

The most significant findings of the survey have been summarized and tabulated in tables 16 to 19, inclusive. For convenient reference, this information is divided into four classes—that relating to the family, the home, the farm, and relocation desires.

A study of relocated families made by a sampling procedure in 1936 indicated that 62 percent of the families still resided in the reservoir counties. The ratio of home ownership had increased slightly, mostly due to a change from an agricultural to a nonagricultural status. Those relocating outside the reservoir counties paid an average of $4,500 for their new farms; those relocating in the reservoir counties averaged only $2,000. Farm owners received a median price of $3,000 from the TVA and invested $2,300 in new farms. However, the new farms were smaller, the median falling from 61 acres to 49 acres. All indications pointed to an agricultural overpopulation situation in the Norris counties.

TABLE 16.—*Summary of answers to questionnaire—relative to the family*

	Land-owner	Tenant	Total
Husband and wife living together .... percent..	80.1	93.4	....
Widowed homes .... do....	15.3	4.4	....
Number born in county of present residence:			
Husbands .... do....	....	....	89.5
Wives .... do....	....	....	88.9
Median residence:			
On present farm .... years..	18.5	3.9	....
In present community .... do....	35.5	16.0	....
Median age:			
Husbands .... do....	48.7	34.9	....
Wives .... do....	47.1	30.8	....
Average education:			
Husbands .... do....	....	....	5.6
Wives .... do....	....	....	5.9
Church preference—Baptist .... percent..	70.6	80.1	....
Fraternal membership:			
Men	....	....	486
Women	....	....	None
Median persons in family	4.5	....	4.4
In relief family	....	5.0	....
In nonrelief family	....	3.9	....
Children living at home	....	....	2.6
Median age:			
Boys .... years..	14.1	9.2	....
Girls .... do....	15.8	8.0	....
Normal educational attainment:			
Boys .... percent..	....	....	35.3
Girls .... do....	....	....	45.4
Education 3 or more years retarded:			
Boys .... do....	....	....	33.1
Girls .... do....	....	....	23.4
Life insurance carried by families .... do....	9.1	7.0	....
Accident insurance .... do....	2.3	2.0	....
Families receiving no current literature .... do....	....	....	45.8
Receiving both newspaper and magazine .... do....	....	....	23.5
Persons with city work experience	....	....	240

TABLE 17.—*Summary of answers to questionnaire—relative to the home*

	Land-owner	Tenant	Total
Average rooms per dwelling	4.4	3.1	....
Homes wired for electricity .... percent..	....	....	1.8
Springs as source of water .... do....	....	....	69.9
Median distance from trade center .... miles..	4.4	3.5	....
Average distance from—			
Elementary school .... do....	1.5	1.6	....
High school .... do....	8.1	8.3	....

TABLE 18.—*Summary of answers to questionnaire—relative to the farm*

	Land-owner	Tenant	Total
Median size of farm .... acres..	62.6	11.7	....
Median area cultivated—1934 .... do....	19.5	10.4	....
Median value of livestock	$224	$62	....
Median value of machinery	$54	Small	....
Median value of furniture	$211	$100	....
Possessing automobiles .... percent..	25	17	....
Median age .... year..	....	....	5.5
Median value	....	....	$144
Value of total personal property	$552	$188	....
Mortgages on personal property .... percent..	12	1.8	....
Average cash income—1933:			
Sale of farm products	$205	$62	....
Other sources	$144	$119	....
Cash value of products produced on farm and consumed in household:[1]			
Per family	$309	$228	....
Per capita	$68	$49	....
Average expenditure for seed, fertilizer, machinery, etc.	$98	$26	....

[1] Exclusive of those families producing none for this purpose.

TABLE 19.—*Summary of answers to questionnaire—relative to relocation desires*

	Land-owner	Tenant	Total
Those expecting to continue in present status after relocation ----percent--	93.1	86.7	
Median size farm desired ----acres--	47.6	23.4	
Median land desired for cultivation ----do----	25.7	18.1	
Desiring to continue as full-time farmers ----percent--			70
To combine agricultural and industrial work ----do----			18

ACCESS TO ISOLATED AREAS

The acquisition of reservoir land up to the normal taking line left numerous areas of various sizes which would be severed from the existing road system and to which access would have to be provided. Studies were made of each isolated area, and where the cost of access facilities exceeded the purchase price of the land in the area, it was recommended that such areas be purchased. Some isolated areas which were subject to severe soil erosion or were of a submarginal nature were recommended for purchase even though not economically justified on the basis of cost of access versus cost of land. Many inaccessible areas were studied originally to determine the advisability of purchase. Those areas actually acquired will be considered in chapter 7 of this report. Families in these isolated areas whose land was purchased were given the same considerations in regard to assistance and advice for relocating as were given to the families in the flowage area.

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CHAPTER 3

DAM AND POWERHOUSE DESIGN

The dam consists of a straight gravity concrete structure for the western 1,570 feet which is connected to the east hillside by a rolled earth embankment about 290 feet long, with a reinforced concrete core wall. The maximum base thickness of the dam is 208 feet and the greatest vertical height is 265 feet (lowest point in the dam foundation to the roadway).

The exposed rock formation in the original river bottom at the site lies at an average elevation of 818 feet above sea level, and the roadway on top of the dam is at elevation 1,061. The crest of the three 100-foot spillway openings is at elevation 1,020. Three hydraulically operated steel drum gates are installed along the concrete spillway crests and the crests of the gates may be raised to any elevation between 1,020 and 1,034 to retain flood peaks. A concrete-encased steel girder roadway bridge spans the spillway. There are eight outlet conduits, arranged in pairs, through the dam. Each outlet is controlled by two vertical sliding gates, one for service and one for emergency use. These outlets permit a discharge of approximately 36,500 cubic feet per second with the reservoir at elevation 1,020.

The powerhouse, 69.5 feet wide by 205 feet long, is situated on the east side of the river immediately against the downstream face of the dam and adjacent to the spillway. There are two generating units, each served by an individual penstock. Each unit consists of a 66,000-horsepower vertical Francis turbine directly connected to a 56,000 kilovolt-ampere generator. The current generated is carried through underground cable tunnels to step-up transformers in an outdoor switchyard, located east of the powerhouse.

DAM

STRUCTURAL ANALYSES

In order to test the adequacy of the final design of the dam on the various possible conditions of loading, it was necessary to make extensive analyses and studies of stress and stress distribution, sliding factors and other factors such as earthquake effects. Gravity analyses and twist studies [1] were made by the Bureau of Reclamation, and an independent check of some of the conditions of loading was made by the Authority. In addition the Authority made a series of analyses in order to investigate possible failure by sliding along an inclined plane of weakness in the foundation rock.

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[1] Houk, I. E., Design of Cross Section, Gravity Analyses, and Twist Studies for Norris Dam. U. S. Bureau of Reclamation Technical Memorandum No. 354.

All stress and stability studies presented here apply to the finally adopted design. No references are made to the preliminary design of a somewhat higher dam which was used in choosing the best type of structure or to the preliminary analyses which were made to

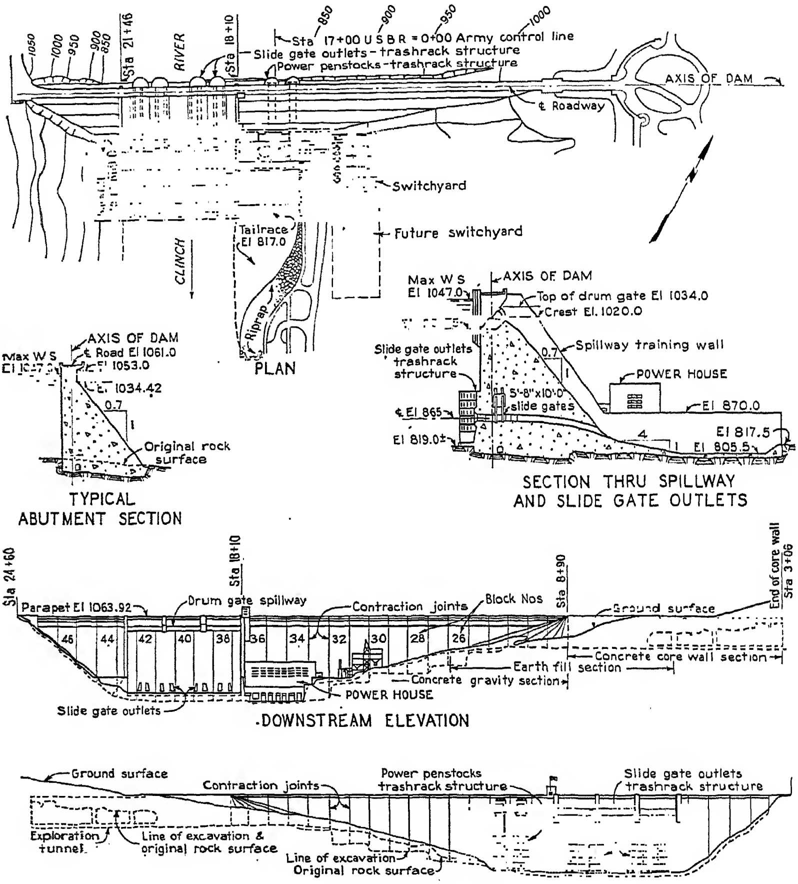
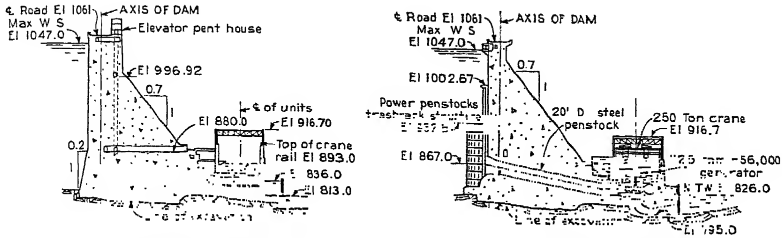


FIGURE 26.—*Plan, elevations, and sections.*

establish the thicknesses of cross section and the upstream and downstream slopes.

The gravity analyses were made on the assumptions that full water load would be carried by gravity action and that the distribution of vertical stress from the upstream to the downstream face of the dam would be according to the straight line theory at all elevations. The trial load twist studies assumed that part of the water

load would be transferred to the abutments by twist action through a system of horizontal beam elements and that the remainder would be transferred vertically to the foundation by gravity action through a system of cantilever elements. The distribution of vertical stresses in the twist studies was assumed to follow the straight line theory.



SECTION THRU WEST END OF POWER PLANT

SECTION THRU POWER PLANT AND INTAKE

FIGURE 27.—*Cross sections through powerhouse*

Basic data.

The design of the cross section was based on an assumed hypothetical reservoir surface elevation at 1,052, five feet above the assumed maximum possible reservoir surface. With this loading the thicknesses of cross section and slopes of upstream and downstream faces of the dam were proportioned so that the sliding factor, for the assumption of two-thirds uplift (as explained under items 3 and 4 of Assumption Factors on page 77), would not exceed 0.65 in any part of the structure except where it was reinforced against horizontal sliding or shear. An example of this condition occurred in the horizontal planes adjacent to the drum gate chambers in the spillway section. Slightly higher sliding factors were permissible during super-hypothetical flood conditions, which were assumed to cause the reservoir surface to rise to elevation 1,060, or during the occurrence of the assumed maximum earthquake. The maximum earthquake was assumed to occur when the reservoir was filled to the top of the drum gates in their raised position at elevation 1,034.

The nonoverflow or abutment sections shown in figure 26 have a base thickness of 203.09 feet at the assumed minimum foundation elevation of 800 feet above mean sea level. In the spillway section, for purposes of analysis, a theoretical 0.70 downstream slope was assumed between the end of the ogee crest curve to the foundation level, and the apron curve near the base of the section was ignored. Thicknesses of cross sections in the spillway section below the end of the ogee crest curve are 7 feet greater than at corresponding elevations in the abutment sections.

The principal items of specified structural data which controlled the final dam design are as follows:

1. Elevation of top of dam, 1,061.
2. Width of top of dam in abutment sections, 20 feet, not including 4-foot overhang.
3. Location of spillway, main river channel.
4. Total length of spillway, 300 feet (excluding two piers).

5. Spillway control, three 100- by 14-foot stabilized drum gates.
6. Elevation of permanent spillway crest, 1,020.
7. Elevation of top of drum gates, raised position, 1,034.
8. Required thickness of concrete on upstream side of drum gate chamber, 15 feet.
9. Radius of the drum gate section of ogee curve, 34 feet.
10. Alignment of vertical portions of upstream face straight and continuous through spillway and both abutment sections.

Assumptions.

The following assumptions were made in the stress and stability analyses:

1. Vertical stresses vary as straight lines from upstream to downstream face of dam at all elevations and at all locations in both spillway and abutment sections of the dam.
2. Stresses at faces of dam act in directions parallel to the slopes of the faces and are equal to the calculated vertical stresses divided by the squares of the cosines of the angles between faces and the vertical.
3. Uplift pressures vary as a straight line from full reservoir pressure at the upstream face of the dam to zero pressure or to tail water pressure, whichever is greater, at the downstream face.
4. Uplift pressures according to the above curve act over two-thirds the horizontal area of the base and over two-thirds the areas of the horizontal concrete sections analyzed at elevations above the base, uplift pressures being assumed to act in the pores of the concrete as well as upon the plane of contact between the concrete and the foundation rock.
5. Maximum earthquake assumed to have an acceleration equal to one-tenth of gravity, a period of vibration equal to one second, and a direction of vibration at right angles with the axis of the dam.
6. Maximum effect of earthquake shocks during the empty condition of the reservoir occurs when the inertia force of the dam caused by the earthquake is acting in an upstream direction.
7. Maximum effect of earthquake shocks during the condition of full reservoir occurs when both the force of the water and the inertia force of the dam caused by the earthquake are acting in a downstream direction.
8. Effects of earthquake shocks during both conditions of reservoir loading assumed to be resisted entirely by gravity action.
9. Shear resistance to any elevation assumed to be increased by the coefficient of friction times the summation of vertical forces acting at that elevation.
10. Lowest foundation level assumed in the analyses to be at elevation 800 in both spillway and abutment sections.

Constants used in stability analyses were:

1. Unit weight of water, 62.5 pounds per cubic foot.
2. Unit weight of concrete, 150 pounds per cubic foot.
3. Modulus of elasticity of concrete in compression and tension, 4,000,000 pounds per square inch.
4. Modulus of elasticity of concrete in shear, 1,667,000 pounds per square inch, reduced to 1,333,000 pounds per square inch to allow for nonlinear distribution of shearing stresses from the upstream to the

downstream face of the dam, the analytical studies being made on the basis of a straight line distribution.

5. Modulus of elasticity of foundation and abutment rock in compression and tension, 4,000,000 pounds per square inch.

6. Ratio of modulus of elasticity of foundation and abutment rock to modulus of elasticity of concrete, unity.

7. Coefficient of friction of concrete on rock or concrete on concrete, 0.65.

8. Ultimate strength of concrete in shear, 400 pounds per square inch.

Design studies.

Analyses of stress distribution, sliding factor, and factors of safety against failure in shear for both spillway and abutment sections were made for the following assumed loads:

1. Hypothetical flood conditions. Full water load carried by gravity action. Reservoir water surface elevation 1,052. Drum gates raised. Tail water surface elevation 847.
2. Reservoir empty. Dead loads carried entirely by gravity action.
3. Maximum flood conditions. Full water load carried by gravity action. Reservoir water surface elevation 1,047. Drum gates raised. Tail water surface elevation, 839.5.
4. Normal full load operating conditions. Full water load carried by gravity action. Reservoir water surface at top of drum gates in raised position, elevation 1,034. Tail water surface, elevation 830.
5. Superhypothetical flood conditions. Full water load carried by gravity action. Reservoir water surface at top of dam elevation 1,060. Drum gates raised. Tail water surface, elevation 859.6.
6. Maximum assumed earthquake shock in time of normal full load operating conditions, elevation 1,034. Period of vibration, one second. Acceleration, one-tenth of gravity. Full water load carried by gravity action. Reservoir water surface at top of drum gates in raised position, elevation 1,034. Tail water surface, elevation 830.
7. Maximum assumed earthquake shock during empty condition of reservoir. All loads carried by gravity action.
8. Hypothetical flood conditions. Full water load carried partly by twist action and partly by gravity action. Reservoir water surface, elevation 1,052. Drum gates raised. Tail water surface, elevation 847.
9. Different uplift conditions during normal full load operation. Full water load carried by gravity action. Reservoir water surface at top of drum gates in raised position, elevation 1,034. Tail water surface, elevation 830.

All of the analyses listed above except (9) were made for the assumption of two-thirds uplift. A discussion of these analyses is given in appendix C.

No analyses of the effects of sand or silt load on internal stress conditions were made, inasmuch as the accumulation of such deposits at the face of the dam would simply mean a slightly increased vertical load on the face with negligible change in any horizontal pressure or total horizontal force.

In order to check further the adequacy of the design, a structural model of the abutment section of the dam was constructed and

tested.[2] A summary of the tests is given in appendix D. The results of stresses obtained analytically and those obtained experimentally from the model are compared in table 20.

TABLE 20.—*Comparison of analytical and experimental stresses*

Elevation	Vertical stress								Stress parallel to face analytical				Maximum principal stress, experimental			
	Analytical				Experimental											
	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
910	-----	-----	-----	-----	95	10	43	83	-----	-----	-----	-----	121	15	62	134
900	179	1(T)	80	98	-----	-----	-----	-----	179	1.6(T)	80	146	-----	-----	-----	-----
870	-----	-----	-----	-----	211	25	120	110	-----	-----	-----	-----	215	38	120	168
850	211	12	88	140	-----	-----	-----	-----	219	18	91	209	-----	-----	-----	-----
800	233	33	99	183	-----	-----	-----	-----	243	49	103	273	-----	-----	-----	-----
797	-----	-----	-----	-----	165	24	56	85	-----	-----	-----	-----	176	44	81	191

1. Dead load stress at upstream face.
2. Dead load stress at downstream face.
3. Combined water load and dead load stresses at upstream face.
4. Combined water load and dead load stresses at downstream face.

For analytical stresses see figure 321, appendix C.
Experimental stresses were obtained from strain measurements on model (see appendix D).
All stresses are in pounds per square inch.

Foundation.

Stability studies[3].—In the gravity analyses the foundation under the dam was not taken into consideration except as to the implied assumptions that the foundation is equal to or stronger than the concrete of the dam. It was thought, however, after considerable study, that such might not be the case due to the stratification of the rock underneath the dam which dips slightly downstream. The Authority, therefore, made stability studies of the possibility of sliding in the foundation, assuming that sliding might occur along some plane within the rock more readily than between the base of the dam and the rock or along a horizontal plane in the concrete.

The analysis as to the stability of the possible weaker zone in the foundation rock resolved itself into the determination of the "factor of safety" under this assumption. However, in dealing with the stratified rather than a homogeneous rock it became necessary to determine:

1. The probable "plane of least resistance" under the body of the dam together with the forces resisting sliding and the forces tending to cause sliding along this plane.
2. The rational assumptions as to the manner in which the rock downstream from the dam would fail if sliding of the overlying rock and the dam were actually to commence along the "plane of least resistance."
3. The resistance of the "toe rock" to such sliding.

In this study the cross-section of the spillway and abutment sections, the structural data, and the unit weights of water and concrete were the same as those used for the gravity analyses and twist studies.

[2] Smith, Eldred D., Tests on Plaster-Celite Model of Abutment Section of Norris Dam. U. S. Bureau of Reclamation Technical Memorandum No. 433.

[3] Based on methods proposed by Col. F. W. Scheidenhelm, a member of the Authority's Board of Consulting Engineers.

The weight of rock was assumed at 160 pounds per cubic foot. The conditions and assumptions used for the calculations were:

1. The most severe conditions within the limits of probability.
2. The most severe possible conditions within the limits of reason.
3. Assumptions arrived at after experiments had been made to check the propriety of assumptions (1) and (2).

The assumed values taken for these three conditions are given in table 21.

TABLE 21.—*Assumed values for stability studies*

	Assumptions		
	Condition 1	Condition 2	Condition 3
1. Coefficient of internal friction	0.70	0.60	0.75
2. Pure shearing resistance in "planes of sliding" under dam (i. e. approximately parallel to the bedding plane) pounds per square inch	500	300	500
3. Pure shearing resistance in "planes of shearing" in toe rock (i. e. across bedding) pounds per square inch	600	400	600
4. Proportion of intact rock in "planes of sliding" under dam....percent..	45	35	35
5. Proportion of intact rock in "planes of shearing" in toe rock......do....	75	50	50
6. Dip, downstream, transverse to axis of dam..........degrees..	5	3	3
7. Angle of rupture of toe rock with horizontal to provide minimum resistance..........degrees..			33

Tests to determine the shearing strength of the rock indicated as a reasonable minimum value,

$$S=1{,}100+0.85q$$

in which S is the unit shearing strength in pounds per square inch resisting any movement, 1,100 is the pure shearing strength of the material in pounds per square inch, and q is the unit compressive load in pounds per square inch exerted transverse to the plane of shearing. The factor 0.85 is the coefficient of internal friction.

In view of these tests, it was apparent that condition No. 2 was too severe and was in fact somewhat beyond the bounds of reason. For this reason, condition No. 3 was developed and is believed to show the most severe conditions which could reasonably be applied to the foundation of the dam in the light of experimental evidence.

The reductions in pure shearing resistance from 1,100 pounds per square inch to 500 and 300 pounds per square inch, respectively (see table 21), were made to allow for the fact that the test pieces may have been stronger than the rock as a whole because the test pieces were large pieces of core remaining after the drilling had broken up some of the other material. The value of coefficient of friction is increased in condition No. 3 because the tests previously referred to seem to justify this increase.

In addition to the above data the following conditions were also considered in these computations:

1. Reservoir water at elevation 1,052 and at elevation 1,060.
2. Corresponding tail water at elevation 847 and elevation 859.6.

Vertical pressures acting on an assumed plane of resistance were made up of the weight of the concrete in the section, the weight of water acting on the inclined portion of the upstream face, the weight of the rock under the dam, and the weight of the tail water acting on the dam and on the downstream portion of the rock at the

toe of the dam. A negative vertical pressure is also present due to uplift.

The results obtained from the analysis of assumption No. 3 give values of factor of safety 4.97 and 4.90 for the abutment and spillway sections, respectively, with reservoir water at elevation 1,052 and tail water elevation 847. Sample calculations for these analyses are included in appendix C.

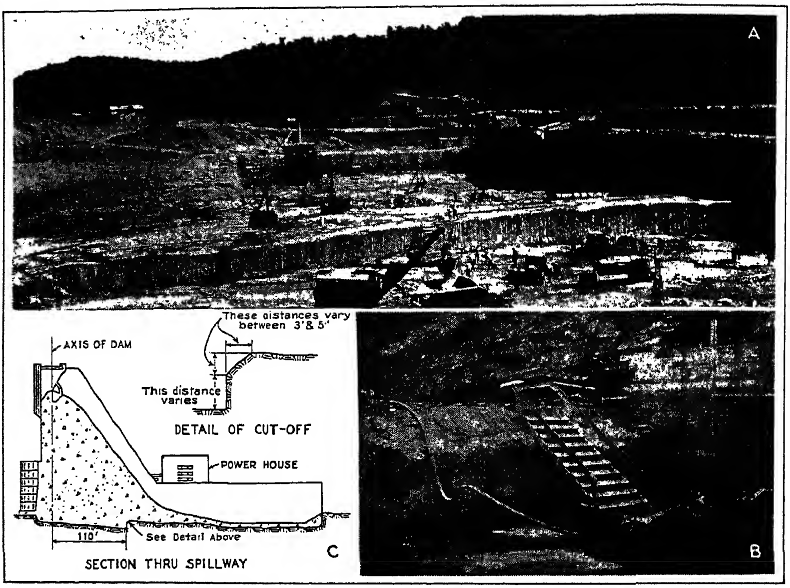


FIGURE 28.—*Treatment of step in foundation.*

Step in foundation.—As the investigation of the rock and foundation conditions progressed during excavation and in the 36-inch core holes, it was found that certain seams in the foundation gave indications of offering less resistance in sliding than was desirable. It was also believed that the joint between concrete and rock offered more resistance to horizontal movement than the seams in question. One particularly unfavorable condition existed at a seam in the rock about 10 feet below the rock surface to which the original excavation was made. The consulting board reviewed this condition and decided that this seam could not be effectively treated by grouting and should be removed. The rock was therefore excavated down to the seam between the side lines of the spillway and a line 110 feet downstream from the axis of the dam.

The effect of the resulting "step" in the foundation rock, being rather unusual in dam foundations, was investigated[4] and was found to

[4] Waldorf, W. A., Memorandum to Chief Designing Engineer, U. S. Bureau of Reclamation, July 16, 1934. Houk, Ivan E., Memorandum to Chief Designing Engineer, U. S. Bureau of Reclamation, July 17, 1934.

present a source of stress concentration in the concrete which was not altogether desirable.

In view of the findings of this investigation, the top edge of the "step" was roughly rounded as shown in figure 28 in order to reduce the expected concentration of stress in the re-entrant angle. In addition a considerable amount of reinforcing was inserted in a plane normal to the greatest stress to minimize cracking of the concrete.

Grouting.—The general plan for grouting the foundation rock under the dam provided for low pressure grouting over the entire area followed by high pressure deep curtain grouting.

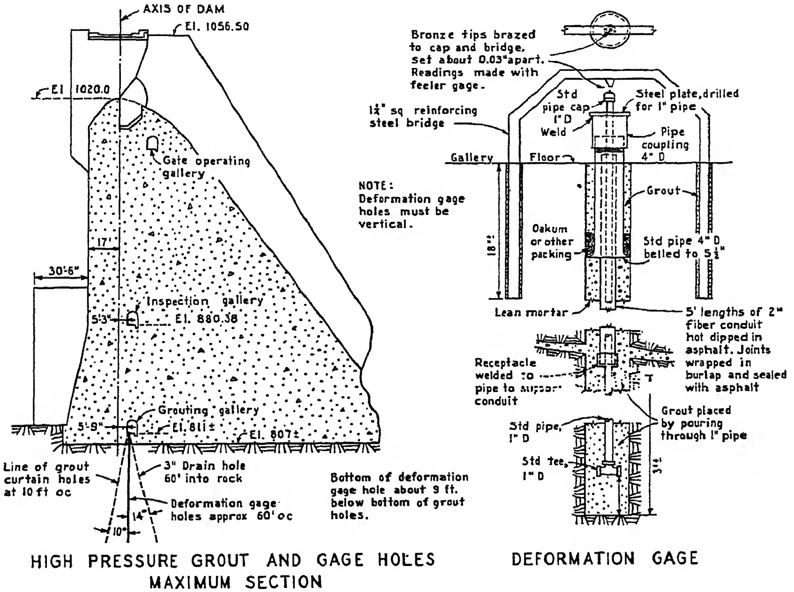


FIGURE 29.—*Foundation deformation gage.*

Prior to the placing of any concrete in the dam the design provided that the near-surface rock of the foundation under the main dam be grouted by means of the shallow holes 20 to 30 feet in depth. The purpose of this relatively shallow grouting was to secure general consolidation in the upper layers of the rock so that deeper penetration and more effective sealing might be secured with higher pressures in grouting the main cut-off curtain. The actual number and spacing of the holes and pressure to be used for the grout injections were determined by the nature of the shallow depth rock as revealed by excavation and the results of water pressure or the tests and the grouting operation.

The main cut-off or grout curtain was provided by the high pressure grouting of deep holes drilled at 10-foot centers on a line 5.75 feet downstream and parallel to the axis of the dam, except in a sec-

tion of the west abutment where the holes were located slightly upstream from the axis. These holes, with an upstream dip of 80° from the horizontal were drilled from a gallery placed as close to rock as practical and to a depth of 70 feet under the maximum section increasing uniformly to 200 feet at the ends of the abutment sections. In the earth fill at the east end the holes were immediately under the cut-off wall.

Prior to drilling the deep curtain grout holes, vertical holes on 60-foot centers on the same line as the deep grout holes were drilled to a depth approximately 10 feet deeper than the grout holes. Deformation gages (see fig. 29) were installed in these holes for the purpose of observing vertical movement of the foundation due to the grouting operations. The information was used in deciding limits for grouting pressures that could be applied with safety. A detailed discussion of the grouting program is given in chapter 6.

Drainage.—For drainage, the design provided for a series of holes at 40- to 100-foot centers on a line 10.75 feet downstream from and parallel to the axis of the dam, inclined so as to drain the foundation on the downstream side of the grout curtain zone. Drilling was done from the grouting and drainage gallery, and in general to a depth slightly less than the deep grout holes.

The drainage holes were not drilled until all grouting within a distance of 150 feet had been completed. The depth of the drainage holes in general was slightly less than the deep grout holes.

The drainage system under the spillway apron and the left training wall consists of 24-inch diameter bell and spigot half-circle concrete gutter pipe laid in an inverted position on bedrock. The lines of pipe are perpendicular to the axis of the dam and spaced on 28-foot centers midway between contraction joints of the apron. These are connected to a transverse header of 36-inch half-circle gutter pipe located 3 feet downstream from the contraction joint between the dam and the apron. Two 24-inch diameter cast-iron pipes located next to the training walls drain the transverse pipe into the spillway apron.

The area immediately behind the west training wall is drained by two lines of 8-inch semi-circular drain pipe located along the heel of the wall and covered with a 12-inch blanket of gravel. A 12-inch outlet through the downstream wall is provided at elevation 813.5.

Tests on foundation rock.—Tests [5] of rock cores taken from the foundation included determination of the specific gravity, absorption, saturation under high hydrostatic head, elastic properties, and compressive strength. Table 22 shows the result of these tests. A comparison of the values obtained from similar tests on cores of the foundation rock of Boulder and Grand Coulee Dams with the values obtained from the Norris cores is shown in table 23. These tests indicate that from a standpoint of strength and elastic properties the foundation rock is adequate to withstand the load which will be imposed in service.

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[5] Price, W. H., and Vidal, E. N., Progress Report, Tests of 4⅜-inch Diameter Dolomite Rock Drill Core and 17-inch Diameter Concrete Drill Cores from Norris Dam Site. U. S. Bureau of Reclamation Technical Memorandum No. 481.

TABLE 22.—*Results of tests on foundation rock*

Core No.	Dry state					Specific gravity	Soaked 25 days						Under 690-foot head 7 days						Type break
	S	E/10⁶		Poisson's ratio			S	E/10⁶		Poisson's ratio		Percent absorption by weight	S	E/10⁶		Poisson's ratio		Percent absorption by weight	
	Pounds per square inch	3,000 pounds per square inch	10,000 pounds per square inch	3,000 pounds per square inch	10,000 pounds per square inch		Pounds per square inch	3,000 pounds per square inch	10,000 pounds per square inch	3,000 pounds per square inch	10,000 pounds per square inch		Pounds per square inch	3,000 pounds per square inch	10,000 pounds per square inch	3,000 pounds per square inch	10,000 pounds per square inch		
A-1	30,400	7.2		0.13		2.74													Cone.
A-2		5.8	7.7	.17	0.23	2.70	17,200	4.0	6.2	0.11	0.21	0.90							Do.
B-1		5.6	7.7	.10	.16	2.80		4.3	7.0	.08	.14	.40	36,950	4.2	6.8	0.10	0.16	0.48	Exploded.
B-2	37,100	5.5		.12		2.77													Do.
C-1		5.9	8.4	.11	.18	2.75	33,700	4.0	6.7	.07	.15	.54							Cone.
C-2	33,000	5.2	7.6	.12	.19	2.72													Exploded.
C-3		5.0	8.0	.10	.20	2.77							21,700	4.0		.12		.32	Wedge.
D-1		5.9	8.1	.11	.16	2.78	27,450	4.6	7.2	.09	.17	.37							Cone.
D-2	25,300	5.2	7.6	.14	.20	2.75													Exploded.
E-1		6.1	8.3	.12	.16	2.76		4.7	7.1	.08	.14	.47	27,250	4.5	7.0	.13	.18	.55	Cone.
E-2	28,450	5.8	8.0	.10	.16	2.78													Exploded.
F-1	28,100	5.3		.12		2.79													Do.
F-2		5.0	7.4	.08	.14	2.80													Do.
G-1	22,800	3.8	5.7	.13	.20	2.77							25,050	4.7		.13		.26	Do.
G-2		3.4	5.5	.06	.13	2.79	28,000	2.8	5.1	.08	.15	.47							Cone.
Average	29,300	5.4	7.5	.11	.18	2.76	26,550	4.1	6.6	.09	.16	.52	27,700	4.5	6.9	.12	.17	.40	

NOTE.—Specimens were in good condition except (A-2) which had a small vertical crack running half way down the core. (G-1) and (G-2) were light gray in color; the rest were dark gray

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TABLE 23.—*Comparison of foundation rocks—Norris, Boulder, and Grand Coulee*

Dam	Type rock			Ultimate compressive strength in pounds per square inch			Modulus of elasticity in millions of pounds per square inch			Poisson's ratio			Stress in pound per square inch of Secant
Boulder	Breccio	4¾x9½		9,260	15,920	22,100			7.5	0.19	0.24	0.38	2,000 to 3,000
Grand Coulee.	Granite	4¾x9½		5,280	15,500	29,700							1,000 to 2,800
Norris	Dolomite	4⅜x8¾	15	17,200	28,150	37,100	3.4	5.4					3,000

NOTE.—Boulder Dam is located on the Colorado River between the States of Arizona and Nevada. Complete data on these cores are recorded in Technical Memorandum No. 304 of the Bureau of Reclamation. Grand Coulee Dam is located on the Columbia River in the State of Washington. Complete data on these cores are recorded in Technical Memorandum No. 459 of the Bureau of Reclamation.

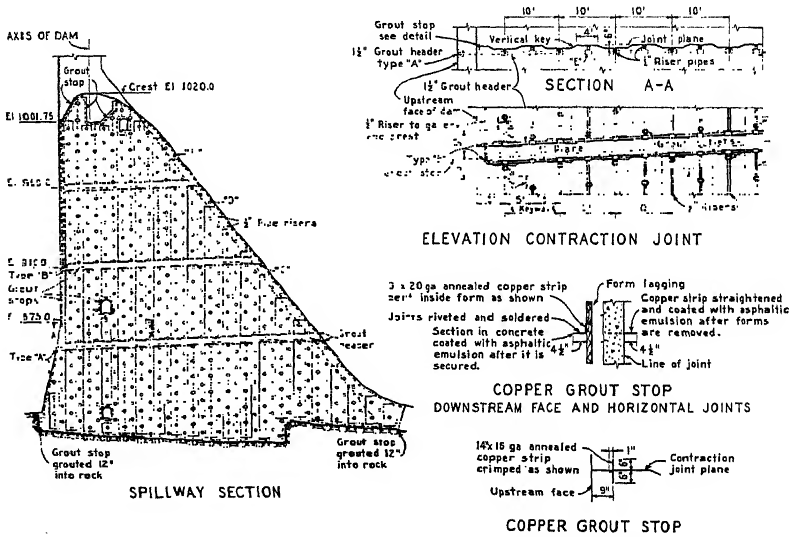

FIGURE 30.—*Contraction joint grouting system.*

Contraction joints.

The concrete section of the dam is divided into 28 separate monoliths (see fig. 26). The joints that divide the dam into these blocks are vertical and normal to the axis of the dam. They also served to divide the dam into convenient working units during construction. The primary objective in providing the joints, however, was to allow for the normal shrinkage due to cooling as the heat of hydration of the cement was dissipated from the structure. In this manner the cracks which would naturally have formed in the monolith due to cooling shrinkage were controlled as to location and shape. No

longitudinal joints were provided in the structure for transverse shrinkage.

In the spillway apron section contraction joints are provided which divide the apron into blocks either 50 or 55 feet long and 28 feet wide. No provisions were made for grouting the apron joints.

Grouting arrangements.—The design provided that when the structure reached an internal state of temperature stability and all the shrinkage due to cooling and other factors causing shrinkage had taken place, the joints would be filled with cement grout and the dam would then approach a monolithic structure. The plan contemplates introducing the cement grout into the joints under pressure through grout cells to which the grout would be conducted through an arrangement of piping embedded in the concrete. The piping terminates in the downstream face of the dam where connections will be made to grout pumps and mixers for introducing the grout into the joints. Each joint is divided into sections by the introduction of copper grout stops placed at intervals of 50 feet vertically and which join the grout stops at the downstream face and the water stop at the upstream face, thus dividing each joint into isolated areas 50 feet high. Grout stops were also provided around all openings such as the galleries which cross contraction joints. This arrangement is provided so that the grouting procedure can be conveniently controlled in definite areas as the grouting proceeds.

Details of installation.—The arrangement and spacing shown in figure 30 provides one cell for each 50 square feet of joint, and is typical for all joints. Minor variations from the set plan occurred to accommodate special shapes or features of the structure. In general, two 1½-inch headers were provided in each 50-foot section, one about 2 feet 9 inches above the bottom grout stop and the other about 1 foot below the top grout stop. The headers are connected by ½-inch vertical risers except that some of the vertical risers are terminated in recesses where they intersect the sloping downstream face of the dam. The two 1½-inch headers are also terminated in the downstream face of the dam. The ½-inch risers are connected to the cells as shown in the detail on figure 31. The headers and the grout stops were placed to follow the 5 percent slope of the horizontal construction joints.

The grout cells consist, as shown in figure 31, of two electric conduit outlet covers placed with the flanges matching. The half cells (one outlet cover) are embedded in adjacent blocks and anchored so that upon opening of the joint the half cells will separate and allow the grout to escape into the joint.

The vertical water stops placed 9 inches from the upstream and downstream faces of the dam span the joints between blocks. The upstream water stop is made from United States standard No. 16-gage annealed copper 14 inches wide crimped so that 6 inches of the strip is embedded in the concrete of each block and a 1-inch crimp is provided to allow movement of the blocks. The grout stops, located horizontally and 9 inches from the downstream face, respectively, are made of United States standard No. 20-gage annealed copper strips 9 inches wide. Both the water stops and grout stops are connected by means of riveting and soldering.

Provision is made for grouting at the abutments and other places where the rock-to-concrete joint lies approximately normal to the axis

and is inclined sufficiently to allow the formation of a shrinkage separation at the contact. Water stops at the upstream face and grout stops at the downstream face were embedded in rock and concrete across the contact. Grout cells were provided by concreting half of a cell with nipple attached into a drill hole in the rock approximately 6 inches deep to which the other half of the cell was attached. The outer half of each cell is attached to the grout piping system.

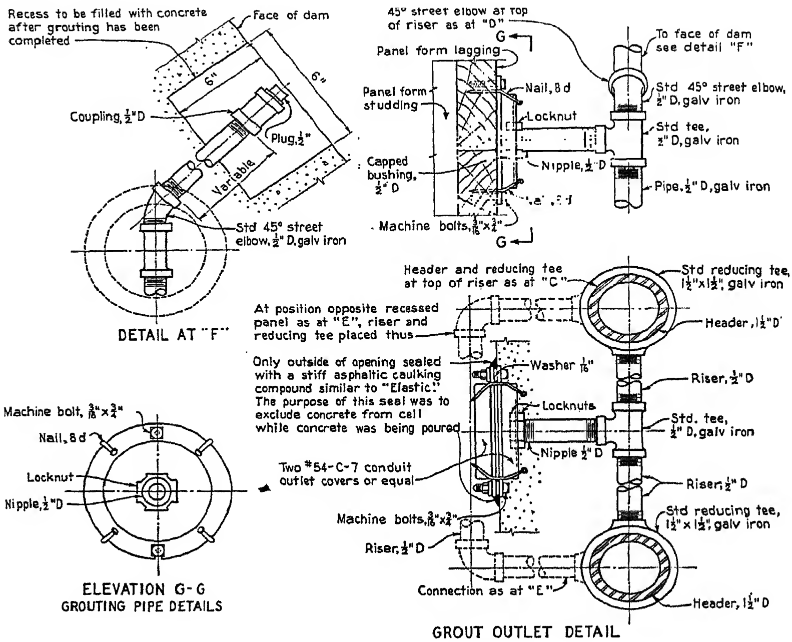

Figure 31.—*Grout cell details.*

Grouting specifications.—The contraction joints are designed to be grouted in separate lifts as provided by the horizontal sealing strips. Tentative specifications for grouting prescribe the following method:

> Just before grouting any section of a joint the section to be grouted and all other sections of ungrouted joints at the same level in the dam shall be filled with water which shall be applied so that the water level in all joints is kept at approximately the same elevation during the filling. During grouting the water level in the joints shall be maintained at approximately the elevation of the top of the section being grouted. The grout shall be pumped into the bottom header of the system of the section forcing the water out of the joint ahead and above the grout. The outlet end of each pipe shall be left open until grout of proper consistency for retention in the joints begins to flow from it, whereupon it shall be capped. The pressure applied to the bottom header shall be such that the resulting pressure at the top of the section being grouted will be approximately, but not in excess of, 50 pounds per square inch. The required pressure will be maintained after the system ceases to take an appreciable amount of grout by means of a stop-cock in the header pipe or other suitable device.

Time of grouting.—The best time for grouting will be determined from observations of the temperature conditions within the dam as indicated by the thermometers embedded in the mass for the purpose of temperature studies. Preliminary studies indicate that this grouting should probably be done during the spring of 1940.

Special investigations.[6]

A program was initiated at the beginning of construction to investigate temperatures, strains, contraction joint openings, deflections, hydraulic uplift, and other characteristics in the dam and foundation in connection with the construction and the operation of the structure in service. Final results of these investigations will be presented in a later report when more complete data are available. Only a brief description of the general program will be included here.

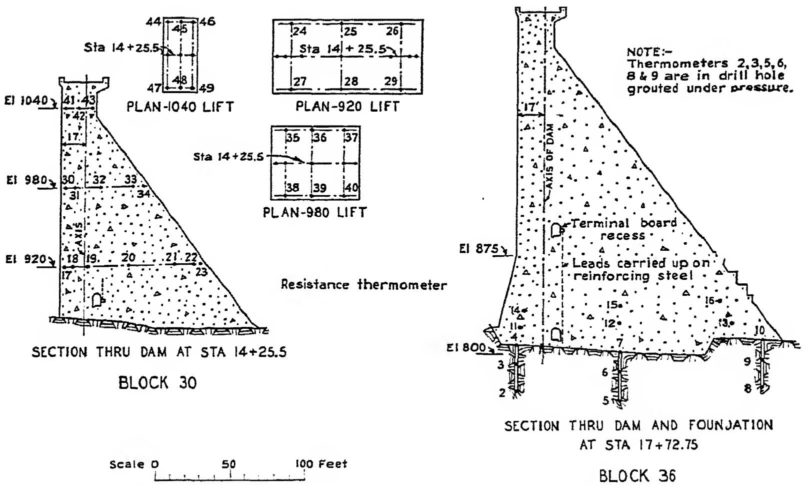

FIGURE 32.—*Typical installation of thermometers.*

Temperatures.—Temperatures at various locations in the dam are measured by Carlson resistance thermometers cast in the concrete or grouted in the drill holes of the foundation. Three-conductor lead wires from each thermometer are brought to terminal boards at convenient locations in galleries of the dam. Resistance measurements by a Wheatstone bridge of special design connected to the thermometer leads furnish data for computing temperatures of thermometer and adjacent concrete. A typical installation of thermometers in the dam and foundation is shown in figure 32.

Strains.—Strains in the concrete are measured by Carlson elastic wire strain meters embedded in the concrete. Two construction blocks of the dam are being investigated near the foundation for

[6] For further details see article by Douglas McHenry and Roy W. Carlson, Measuring Dam Behavior, Engineering News-Record, 122: 440–442, March 30, 1939.

distribution of stress. Three stations with 12 meters at each station were installed in each block as shown on figure 33. Meters are installed in duplicate. "No stress" meters are installed to furnish data on length change due to such properties as moisture and temperature of unstressed concrete. The installations, as shown, furnish data for computations of strain in a plane at right angles to the axis of the dam. Both three- and four-conductor lead wires are used for the strain meters, three-wire leads for meters to compute the strain only, and four-wire leads for meters to measure both strain and temperature.

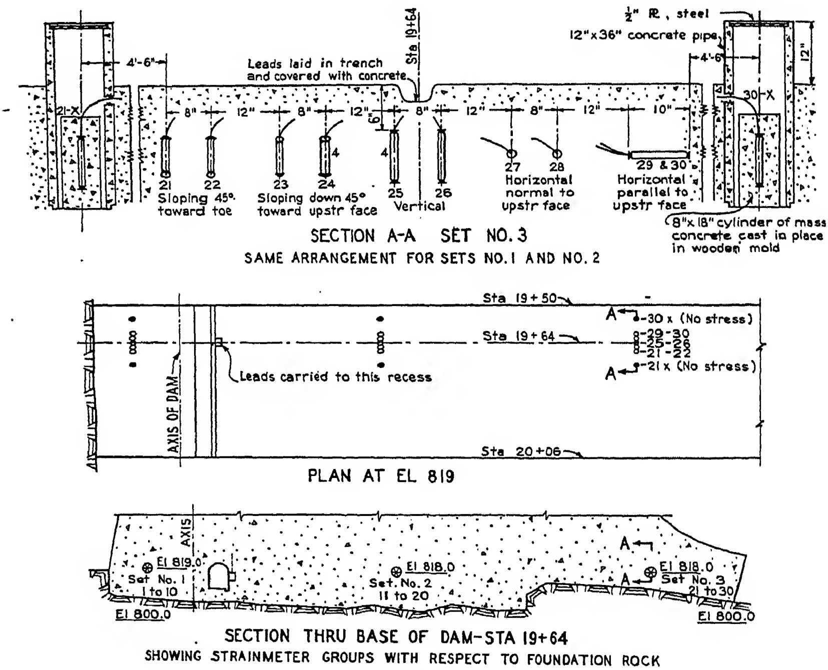

FIGURE 33.—*Typical strain meter installation.*

With proper changes in connections, strain meters are read by the same Wheatstone bridge used for thermometers. Operation of the meter, as the name suggests, depends on change in elongation of elastic wire being proportional to its change in electrical resistance.

Contraction joint openings.—Contraction joint openings are being measured with elastic wire joint meters similar in construction to the strain meters. One-half of the meter is cast in each block separated by the joint to be measured, and change in the length of the meter is used as a measure of the joint opening. The meters have a range of approximately one-fourth inch and a minimum reading of about 0.0007 inch. Three- or four-conductor leads are used as for the

strain meters depending on whether the joint opening only or both joint opening and temperature are desired. The same bridge is used for reading joint meters as for thermometers and strain meters.

Deflection.—Deflection of one block of the dam is being measured by a plumb line suspended in a vertical shaft connecting the 1,051 gallery, the 880 gallery, and the 811 gallery, in block 35. Provisions were made for the accurate reading of the plumb line at the 880 gallery and at the 811 gallery; the top of the line, at the 1,051 gallery, remains fixed. A movable micrometer head microscope and fixed glass scales permit observing the location of the 1/64-inch stainless steel wire plumb line to within 0.0005 inch. Horizontal measurements both parallel and at right angles to the axis of the dam are made. The weight attached to the free end of the plumb line is suspended in a small tank of kerosene to dampen vibration. The line is entirely protected from external air draft except at the small openings used for making observations.

Results to date indicate that temperature deflections far overshadow the load deflections, change in deflections between winter and summer being at least 0.45 inch for the section of the dam investigated. During initial filling of the reservoir in the spring of 1936, the top of the dam actually deflected upstream 0.3 inch due probably to temperature changes.

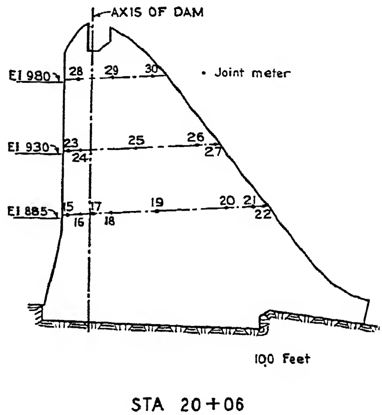

FIGURE 34.—*Contraction joint meter installation.*

Hydraulic uplift.—Provisions are made for measurement of hydraulic uplift pressures at various locations in the mass of the dam and at the foundation. The details of cells and piping and location are shown in figure 35.

HYDRAULIC FEATURES OF DAM

Spillway.

The spillway was located in the river channel. This arrangement permits the use of the hydraulic jump pool at a favorable location and results in the safest and most economical device for dissipating the energy of the spillway discharge without destructive erosion. The channel of the spillway, confined between two concrete training walls, is upon the downstream face of the spillway section of the dam proper and the hydraulic jump apron.

The discharge capacity of the spillway (drum gates down) with water surface at elevation 1,034 is 54,000 cubic feet per second. An additional discharge of 38,500 cubic feet per second with water surface at elevation 1,034 is obtained by means of the eight slide gate outlets. Calibration curves for the spillway gates were secured

through model experimentation.[7] Details of the gate calibration model studies are described in appendix D.

Drum gates.—The reservoir level above the normal crest of the spillway (elevation 1,020) is controlled principally by drum gates located at the top of the spillway section. Slide gates regulate the flow through the outlet conduits and thus control the reservoir level. The drum gates are for the control of the reservoir level at flood stages and to provide 517,000 acre-feet of flood storage between the crest elevation and the top of gates in a raised position. In addition there is an uncontrolled flood storage from elevation 1,034 to elevation 1,052, or an additional 800,000 acre-feet.

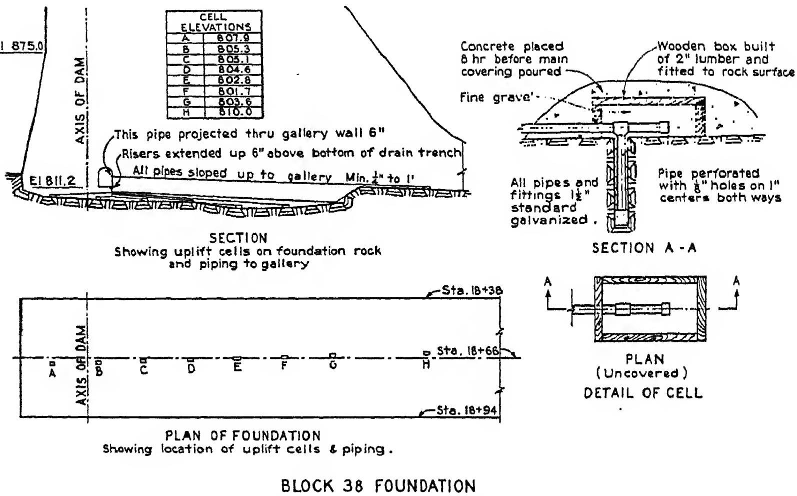

FIGURE 35.—*Uplift measuring equipment.*

The drum gate installation consists of three separate complete units, each 100 feet long and 14 feet high when raised to the closed position. Each unit consists of a structural steel buoyant drum in the shape of a modified section of a cylinder of approximately 15-foot radius and which is hinged along the upstream side to the dam; a chamber for the drum, recessed in the concrete spillway crest; and a piping system with valves, controls, and indicators for regulating the water level within the chamber and thereby the position of the gate. In the open or down position the drum forms a part of the spillway crest.

Forty-three cast steel hinge units support the rolled bronze hinge pins. Each casting is anchored to the dam by four bolts embedded in the concrete. The hinges were designed to provide for temperature expansion of the drum. One end hinge pin, keyed to the drum and extended through the wall, is connected to the gate-control mechanism.

[7] Thomas, Charles W., Hydraulic Model Experiments for the Design of the Norris Dam, U. S. Bureau of Reclamation Technical Memorandum No. 406.

A gate seat of semisteel [8] castings is provided along the downstream side of the gate chamber to support the drum when in the down position and to limit the upward movement. These castings are anchored to the dam by 50 bolts.

Built-up structural steel girders form the framework of the intermediate sections of the gate. End sections are made from a combination of I-beams, channels, and buckle plates. Skin plates ½ inch thick are riveted to the girders to form the covering of the gate. The joints

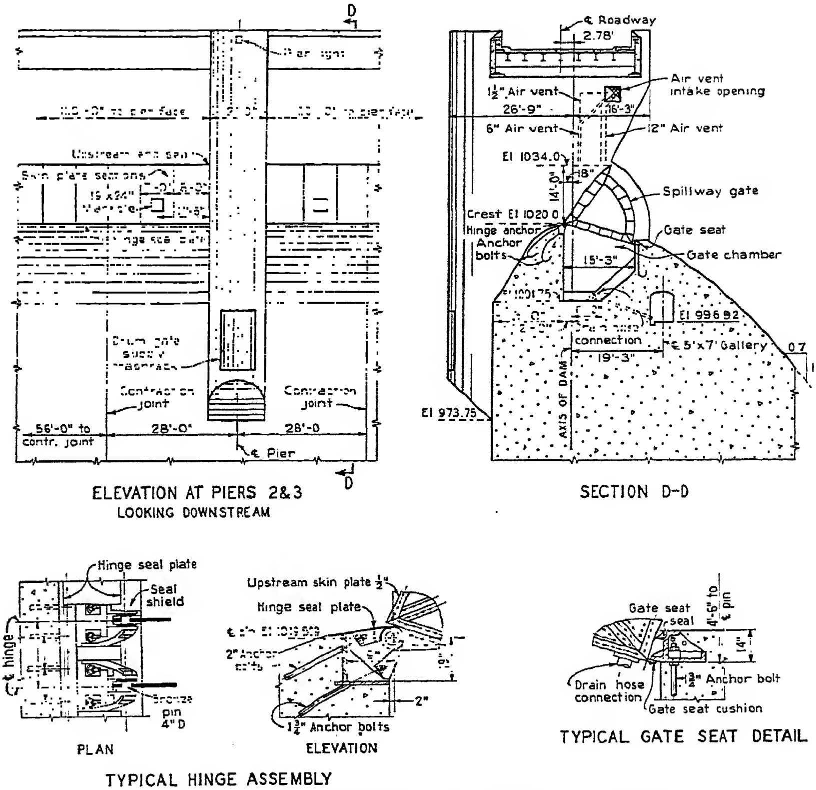

FIGURE 36.—*Drum gate, general arrangement and details.*

between plates are welded for watertightness. The downstream skin plate rivets are countersunk and ground flush with the plate. The welds are also ground flush.

Gate seals are provided to prevent loss of water from the well during operation. The upstream seal is effected through contact be-

[8] Semisteel—High-Test Gray Iron Castings, Federal Specification QQ–I–656.

tween a fixed plate attached to the hinge anchor and a movable plate concentric with the hinge pins and attached to the gate. The downstream seal is fastened to the gate seat and is in contact with the downstream face of the gate. An upper filler seal contacts the top of the gate seal and seals the gate in the down position. End seals contact the concrete piers at the ends of the gate and prevent leakage past the gate, and packing glands prevent passage of water into the operating chamber. It was originally intended that semisteel pier plates be used to provide smooth and true contact surfaces for the end water seals of the drum gates. Because of the infrequent use of the gates and the smooth concrete surfaces obtainable in construction it was felt that the pier plates could be omitted. The surfaces of the concrete piers are true and smooth, and operating results have shown that this omission was justified. The gates have been operated under an 11-foot head of water with no indication of leakage past the end seals. This eliminated 104,000 pounds of cast semisteel plates.

Water for operation of the gates is directed into the gate well from the reservoir through a 48-inch pipe. The intakes for these pipes are formed in the piers, and vertical steel bar trashracks shaped to conform with the piers protect against the entrance of foreign matter. A transition section from rectangular at the trashrack to circular at the upstream end of the pipe is formed in the concrete of the pier. A motor-operated 48-inch gate valve with provisions for manual operation in case of emergency controls the flow of the water into the well. Discharge from the well is effected through a 24-inch line in which is located a 24-inch differential control valve and a 24-inch hand-operated gate valve. A bypass line for emergency operation is also provided in which is located another 24-inch gate valve. Leakage into the drums is drained away through two 4-inch drain holes connecting each drum with a flexible hose to the operating gallery drain trench.

The gates are operated by water in the chambers. Buoyancy of the metal drum causes it to rise or lower with changes of water level of the well. The operating mechanism is designed to provide regulation of the gate crest at any desired position between elevations 1,020 and 1,034.

The control mechanism consists essentially of: a 24-inch differential control valve; a cable which connects a pilot valve of the differential control valve with a weighted anchorage which rests upon an adjustable supporting table; counterweight; two sheaves over which the cable passes between the valve and anchorage, one of which is mounted free to turn on the hinge pin of the gate and the other at the end of an operating arm which in turn is rigidly attached to the gate hinge pin which moves with the gate.

In operation, water enters the gate chamber through the 48-inch line and buoys the gate up. As the gate rises the hinge pin and the operating arm rotate. The upward movement of the operating arm tightens the cable between the weight and the pilot valve on the differential control valve. The gate continues to rise slightly past the desired position and the resulting tension in the cable, acting on the pilot valve, imparts an opening motion to the differential control valve and releases water from the gate chamber. The gate then lowers slightly allowing the valve to close. This cycle is repeated several times, with decreasing motion each time, until the gate reaches a stable position.

The position is maintained as long as the inflow from the reservoir and the discharge are equal. A change in position of the gate is effected by resetting the supporting table for the anchorage. Operation depends on the balance of flow obtained between the intake valve and the differential control valve. The position of the intake valve must be adjusted to a flow within the operating range of the differential valve, account being taken for all leakage at the drum gate seals. The gate can be controlled by means of the hand valve in the bypass line in case of failure of the differential control valve. By using the weighted type of anchorage, the pull of the cable on the pilot valve stem, when fully up, is limited to a safe amount.

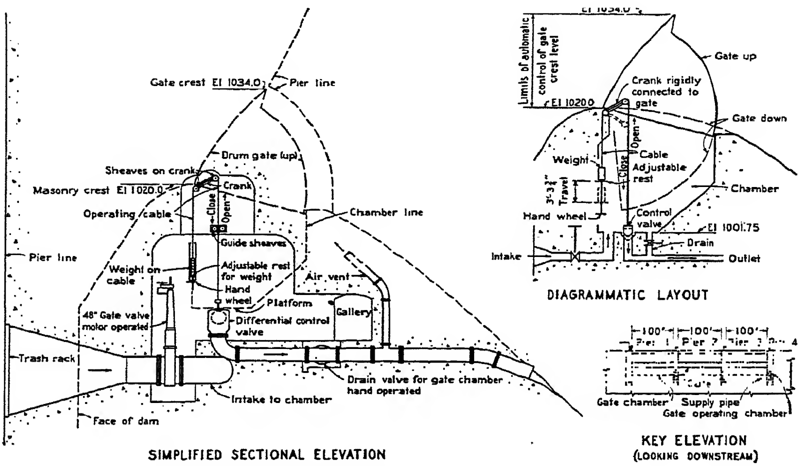

FIGURE 37.—*Drum gate operation details.*

Figure 38 shows discharge rates over one of the spillway gates with the reservoir at different elevations. These curves were developed from laboratory model tests and will be checked by actual discharges over the spillway as recorded during operation of the gates.

The original drum gate design provided a mechanism for automatically moving the gates to hold the reservoir at any desired stage above the spillway crest. Owing to the extensive surface area of Norris Reservoir, local changes in level could be caused by wind and barometric conditions that would be relatively large and without relation to changes in storage volume. Under such conditions extremely erratic gate operation could result from any automatic connection between gate position and reservoir level at a single point such as at the dam.

The control system was therefore redesigned by the Authority's staff for manual operation upon information that would generally be available such as weather forecasts, rainfall and river flow observations, as well as the approximate reservoir stage. An incidental advantage of the modified design was its considerably lower cost as the

result of the elimination of such items as three float wells in the spillway piers, a system of equalizer header piping, a long drain header extending under all three drum gates and down the west training wall, three 57-inch butterfly inlet valves, and a complicated cable control for the differential valves.

The system as installed is entirely manually operated but has stabilizing features to hold the gates at any desired position and has functioned entirely satisfactorily.

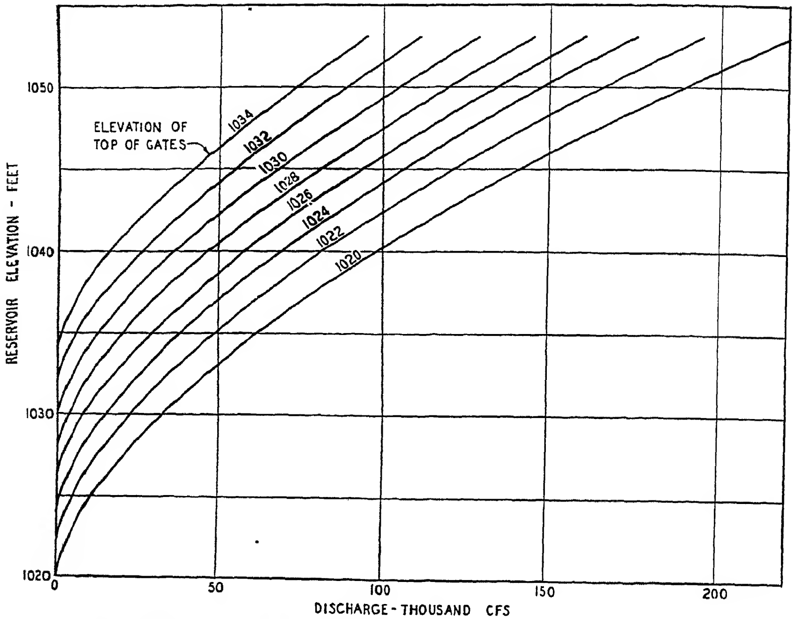

FIGURE 38.—*Drum gate discharge as determined by model studies.*

Training walls.—The training walls on each side of the spillway were designed to withstand differential water loading as determined by model experimentation.[9] The allowable stresses used in the design were 700 pounds per square inch for the compressive stress in concrete, and 16,000 pounds per square inch for the tensile stress in reinforcement bars. The training walls are both constructed of reinforced concrete and are of the same design for the portion extending down the face of the dam from the top to elevation 870. There, the right wall ties into a gravity type wall which extends to the end of the hydraulic jump pool, and the left wall ties into the west end of the powerhouse which forms the section immediately downstream. Downstream from the powerhouse the wall is a reinforced concrete cantilever. The base of the wall on the spillway side forms a part of the spillway apron slab. The wall downstream from the powerhouse was utilized as a

[9] See appendix D.

cofferdam during construction and additional reinforcing steel was placed in the wall to take care of this condition.

Hydraulic jump pool.—The hydraulic jump pool, designed to dissipate the energy of the water flowing over the spillway, is formed by the two training walls and a sill at the downstream end of the apron. The apron slab has a minimum thickness of 4 feet 9 inches and is monolithic with the 12-foot sill. Reinforcing bars grouted into the drill holes anchor the apron to the foundation rock. The final arrangement and details of the size of the jump pool were reached after thorough study by model experiments. These model studies are described in appendix D.

Outlet conduits.

Eight outlet conduits are provided in the base of the dam in the spillway section for discharge of water during low stages of the reservoir and as an added discharge capacity source in time of extreme flood. Trashrack structures are provided to protect against the entrance of foreign matter. The flow through each conduit is controlled by a pair of hydraulically operated slide gates, one for regular operation and the other for emergency use. They are operated from a chamber formed in the concrete of the dam above the conduits. The general design and shape of the outlet conduits were determined by model studies (see appendix D).

The outlet conduits were designed to discharge approximately 36,500 cubic feet per second with water surface at elevation 1,020 and 39,500 cubic feet per second with water surface at elevation 1,047. Past gage records at Clinton indicate that 40,000 cubic feet per second could be passed with little or no damage along the lower Clinch River valley at Clinton and Kingston. The maximum recorded flow (1886) at Clinton is 115,000 cubic feet per second.

Trashracks.—Each pair of outlet conduits is provided with a 24-foot radius semicircular reinforced trashrack structure with vertical steel trashrack bars for protection against entrance of foreign matter into the conduit which might interfere with the proper operation of the gates or conduits. The structures rest on rock at approximately elevation 818 and extend up to elevation 906. Above elevation 843.5, which is the top of the base section of each structure, the trashracks are of beam and column construction. The semicircular reinforced concrete beams, 2 feet 6 inches by 3 feet in cross section, are arranged horizontally with a clear space of 10 feet between beams and supported by reinforced concrete columns spaced to give a clear opening between columns of 6 feet 2¼ inches at the outside periphery of the semicircular structure. In these openings between the concrete beams and columns are placed grill sections made up of 1- by 6-inch flat steel bars spaced 5⅛ inches apart. The top is formed by a reinforced concrete slab two feet six inches thick which receives additional support from three reinforced concrete brackets poured monolithically with the dam. Two openings in the top provide access for stop logs. The trashrack structures are tied to the dam by steel dowels placed in the dam where the beams and the top slab intersect the structure. The stop log grooves are monolithic with the dam.

It was originally planned to use four semicircular trashrack structures, with tops extending to elevation 956. Later and more detailed studies of flow conditions and velocities indicated that a lower height would be satisfactory. The structures were redesigned and built to elevation 906.

Bellmouth sections.—The original design for the inlet end of the outlet conduits utilized a rectangular opening in a vertical plane at the face of the dam. A very slight transition from a rectangular opening of 6 feet 8 inches by 11 feet at the stop log groove to a rectangular opening of 5 feet 8 inches by 10 feet at a point 6 feet downstream from the stop log groove was made along curved lines on a radius of 36 feet 3 inches at the top, bottom, and sides. This design was almost identical with the design for similar conduits of Madden Dam in the Panama Canal Zone. On the basis of Madden Dam experience, a revised design was developed by the Bureau of Reclamation which utilized a bellmouth type of entrance, and the Norris structure was built accordingly.

Conduits.—The eight discharge conduits are located in pairs, one pair in each of four blocks of the spillway section. In plan, one conduit in each pair is straight. The other is straight from the upstream face of the dam to a point about 33 feet downstream from the axis, and from this point the conduits follow a curved line on a 560-foot radius. The profile of all the conduits is level with the center line at elevation 865 for the first or upstream 33 feet, and at that point they curve downward following a parabolic path until the floor meets the slope of the apron of the dam. The conduits are arranged to discharge directly into the hydraulic jump pool. Beginning at a point 6 feet downstream from the upstream end, each conduit is lined for 48 feet with semisteel formed into castings approximately four feet long, each section of conduit lining being composed of two separate castings, one the top half and the other the bottom half. In each conduit two of the assembled castings form the upstream frames, and two more of the assembled castings form the downstream frames for the two slide gates. The gates are so arranged that the remainder of the lining for each conduit is placed with equal length of lining upstream from the upstream gate and downstream from the downstream gate. The 6 feet of each conduit upstream from the lined portion and the remainder downstream are formed in the concrete of the dam.

Gates and operating mechanism.—Each gate consists of a sliding leaf which seats against a frame cast as a part of one of the conduit lining sections, and a 24-inch oil operated hydraulic hoist located directly above the bonnet of the gate. The leaf of each gate is connected to the piston of the hoist with a 6-inch diameter stem. A stem extension is provided above each piston which may be engaged by a suspended semiautomatic gate hanger when the gate is in the raised position. The operating gate and the emergency gate are identical. The original design called for a rather complicated automatic hanger for locking the stems of 8 of the slide gates at any desired position. A semiautomatic hanger which would hold the gates in the fully open position, proposed for 8 of the gates, was used on all of the 16 gates. This hanger consists of two spring-mounted pawls which grip a coned head on the gate stem in the raised position. The pawls may be released for lowering a gate by means of a small cable.

Oil for the operation of the hydraulic hoist is supplied by a single high pressure oil pump. The pump is a rotary gear type, driven by a 20-horsepower motor and has a capacity of 15 to 20 gallons per minute at 1,000 pounds per square inch working pressure when handling oil at temperatures between 50° Fahrenheit and 100° Fahrenheit. It is capable of operating at a maximum working pressure of 2,000 pounds per square inch. The pressure header fittings, pressure header, control piping, control piping fittings, and control valves are all designed for maximum working pressure of 3,000 pounds per square inch. The

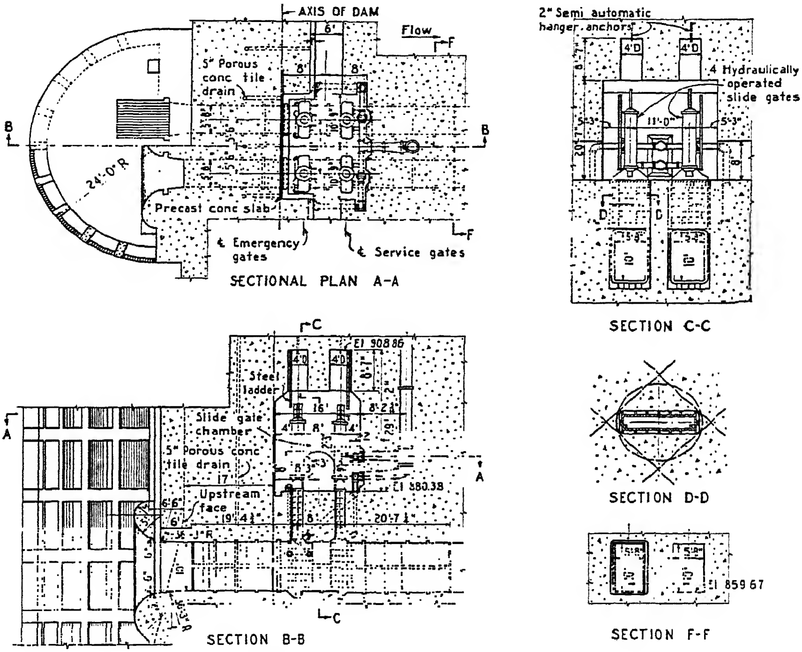

FIGURE 39.—*Slide gate details.*

return piping and fittings are all standard pipe and fittings. The oil storage tank is of arc-welded, plate-steel construction and has a capacity of approximately 300 gallons.

As a rule, only one gate is operated at a time. In the ready-to-operate condition, the entire system is filled with oil. The process of opening or closing the gate is briefly as follows:

1. A four-way valve is turned to a position which will admit oil from the pump into one distribution header and admit oil into the return line from the other distribution header.
2. A shut-off valve and a regulating valve are both opened.
3. Two straight-way valves from the piping to the cylinder of the gate are opened.

4. The pump is started by a "start-stop" push-button station located in the operating chamber.
5. A regulating valve is closed.
6. When the gate reaches the desired position, the regulating valve is opened.
7. The two straight-way valves and the four-way valve are closed.
8. The motor is stopped from the push-button station.

An air-purging system is provided for removing entrapped air from the oil system. An automatic pressure switch located in the pump chamber stops the pump when for any reason during operation a condition arises which causes an excessive operating pressure. A pressure-unloading valve is also provided.

A semiautomatic gate hanger is suspended above each of 16 hydraulic operating cylinders in the slide gate operating chambers. A cap fastened by a safety stud to the top end of the gate stem extension is automatically engaged by the hanger hooks when the gate is raised to the fully open position. The safety stud is so proportioned that it will break if the power is applied to close the gate without first releasing the stem, which is done by a manual operation of the hooks. The diameter, in inches, of the breaking section of the stud is determined by testing a specimen of the stud material and applying the following formula:

$$d=D\sqrt{\frac{w+2000}{L}}$$

in which D = diameter of test piece of prescribed dimensions, in inches,
L = breaking load of test piece in pounds.
w = weight of moving parts of gate in pounds.

The weight of the moving parts of each gate is 13,580 pounds. All studs were turned at the smallest section to a determined diameter of 0.461 inch.

Oil for the operation of the system is Navy symbol oil No. 2190 with the following characteristics:

Viscosity at 130° F	Saybolt 180–220 seconds.
Flash point, minimum	350° F.
Pour point, maximum	35° F.
Carbon residue	0.40 percent.
Evaporation loss, maximum	2.50 percent.

Air-inlet and bypass piping is provided for each set of gates. Two nonslam check valves are placed in the line which serves the upstream gate arranged so that when the downstream gate is closed and the upstream gate is open water is checked from spilling through the air-intake piping downstream from the downstream gate.

The original design for venting the outlet conduits specified eight 12-inch diameter air lines carried upward from elevation 870 to 1,050. This system, in the main, consisted of 3,622 feet of 12-inch Byers wrought-iron pipe, one hundred and thirty-four 12-inch companion flanges, sixteen 12-inch blind flanges, eighty 12-inch cast iron elbows, and eight 12-inch sliding joints. The system was redesigned and simplified using 168 feet of 24-inch ironlined ducts and 250 feet of 30-inch formed ducts for two risers, together with the necessary

8-inch and 12-inch cast-iron piping, fittings, and check valves to carry the air to the gates from the 24-inch header line at about elevation 884.

EAST EMBANKMENT

The eastern 290 feet of the dam is formed by a rolled earth fill with a reinforced concrete core wall. The core wall extends an additional 294 feet into the east abutment. The formed section of the core wall above the original ground line is 5 feet thick. In order to assure an impervious foundation condition in the rock below the core wall, a tunnel 675 feet long was driven and concreted, following a seam of unsound rock at approximately elevation 965. Two shafts extending from the bottom of the open core wall trench down through rock to the tunnel at elevation 965, approximately 75 feet and 180 feet respectively from the west end of the tunnel, served for excavation and concreting. The former shaft was completely concreted. The latter was concreted to elevation 970, left open from elevation 970 to elevation 1,000, and above elevation 1,000 was concreted with a manway provided in it for further observation of underground conditions. At the bottom of the east end of the core wall another tunnel was carried 65 feet eastward and was concreted with the core wall (see figure 26). In anticipation of possible downstream movement of the embankment section, slip joints were provided at two vertical contraction joints of the core wall, and at horizontal joints between the first and second pours, beginning at the connection between the core wall and the gravity section and extending 403 feet east to the most eastern vertical slip joint. One of the vertical slip joints was provided at the connection between the core wall and the gravity section, and the second is provided 9 feet east of the manway leading to the open shaft.

The trench for the base of the concrete core wall extends east into the hill for a distance of 500 feet from the east end of the gravity section. The trench averages 5 feet wide and extends to sound rock. Beginning at the west end of the trench (the east end of the gravity section cut), the rock averages elevation 965 for the first 180 feet beyond which, after a vertical step-up to about elevation 1,004, the rock slopes up unevenly to elevation 1,050 at the extreme east end of the trench.

The earth fill portion of the embankment consists of suitable clay material from borrow pits in the vicinity of the fill. The material was placed in layers, so that when completely compacted, the layers were not more than 6 inches in thickness. Recently developed methods [10] for controlling compaction of rolled earth fills for dams were specified for checking the degree of compaction obtained in the fill.

The portion of the fill upstream from the core wall is protected by a layer of rock riprap laid on a gravel blanket. At the toe of the fill, a trench 4 feet wide and between 2 and 3 feet deep provides a footing for the toe of the riprap. The gravel blanket consists of ¾-inch to 1½-inch crushed stone from concrete aggregate storage, spread to a thickness of 1 foot over the entire surface of the fill.

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[10] Proctor, R. R., Rolled Earth Dams, Engineering News-Record, vol. 111, pp. 245–248, 286–289, 348–351, 372–376.

Stone from the aggregate quarry, ranging in size from ½ cubic foot to 2 cubic yards, placed to conform approximately to the shape of the surface of the earth fill, completes the riprap.

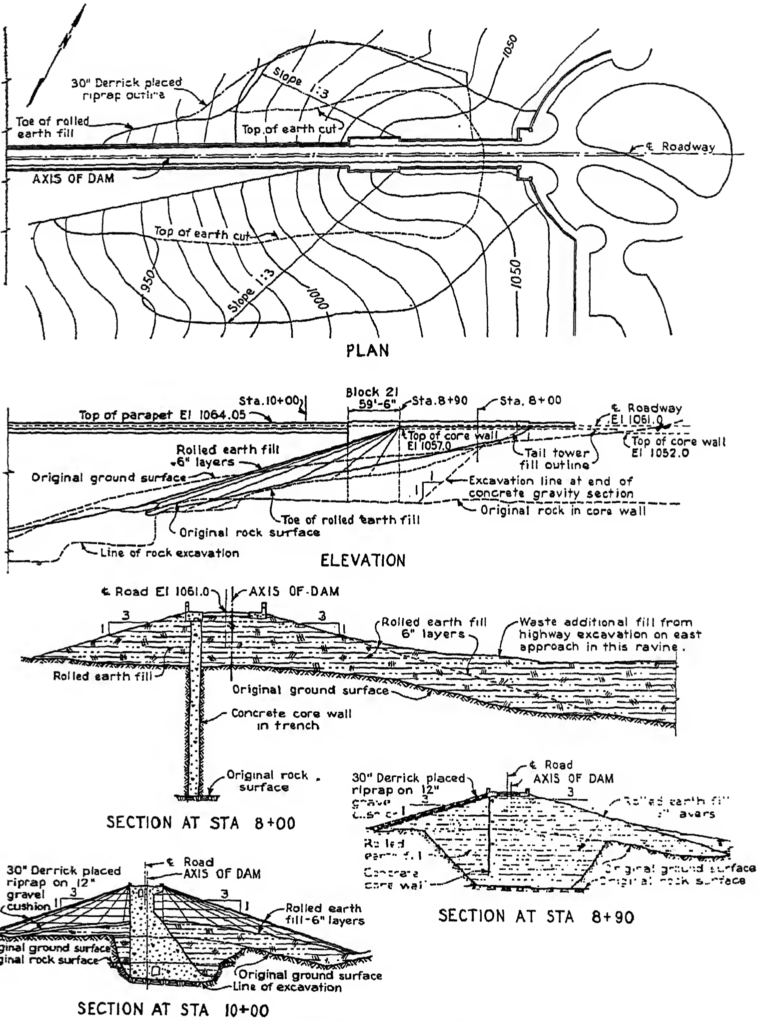

FIGURE 40.—*Details of earth embankment.*

The downstream fill is protected against erosion by a blanket of lespedeza grass. Three drainage ditches at approximately elevation 1,045, elevation 1,010, and elevation 960 serve to intercept the drain-

age from the slope and conduct it either to a point downstream from the fill or to the paved ditch at the intersection of the fill and the downstream face of the dam. The paved ditch at the intersection of the fill and the dam also serves to keep drainage from the face of the dam from eroding the fill. The fill is drained by four lines of 12-inch porous concrete tile approximately 10 feet beneath the original ground level. These empty into a header paralleling the dam which conducts the drainage to the river.

POWER PLANT

The power plant consists of two hydroelectric generating units and the necessary auxiliary equipment. Each unit is composed of a 56,000-kilovolt-ampere generator directly connected to a vertical Francis type turbine rated at 66,000 horsepower. Two penstocks serve to conduct water to the turbines, one penstock serving each unit. The intake entrances of the two penstocks are each closed when not in use by tractor-type gates. An embedded steel scroll case surrounds each turbine, and water is conducted away from the turbines by elbow-type draft tubes.

The main powerhouse structure, 69 feet 6 inches wide by 205 feet long, which shelters the generating units and operating equipment is located adjacent to and immediately downstream from the dam on the east bank of the river. The substructure of the powerhouse, founded on solid rock, is largely mass concrete, and the superstructure is a structural steel framework encased in concrete. The floors are reinforced concrete slabs between concrete encased steel floor beams. Steel trusses set on structural steel columns carry the roof. In addition to the main powerhouse structure, space is provided for housing the control-room equipment and other miscellaneous equipment in a section between the downstream face of the dam and the power units.

POWER INTAKES AND PENSTOCKS

Trashracks.

Each penstock intake is surrounded by a semi-circular reinforced concrete trashrack structure to protect against the entrance of logs or other large trash which might damage the gates or the turbines. Each structure rests on a solid unreinforced concrete semioctagonal base founded on rock. The horizontal members or beams are in the form of streamlined chords connecting vertical streamlined columns spaced 22° 30′ on center. The beams are on 12-foot 6-inch centers vertically and have a clear space between beams of 10 feet. A 24-inch wide flange structural steel beam is provided at each column and anchored to the beam to form slots for the metal trashracks and possible future rack rakes. The top of the trashrack structure is of reinforced concrete slab and beam construction provided with suitable openings for movement of the gate and stop logs. Above the slab, on top of the trashrack structure and extending to elevation 1,002.67, is a rectangular reinforced concrete gate well which serves to guide the gate in traveling from the trashrack structure to the maintenance position on top of this structure and to keep out trash from the deck gate opening. Two structural steel gate support beams are provided for each gate. The beams are attached to the dam and swing out of position when

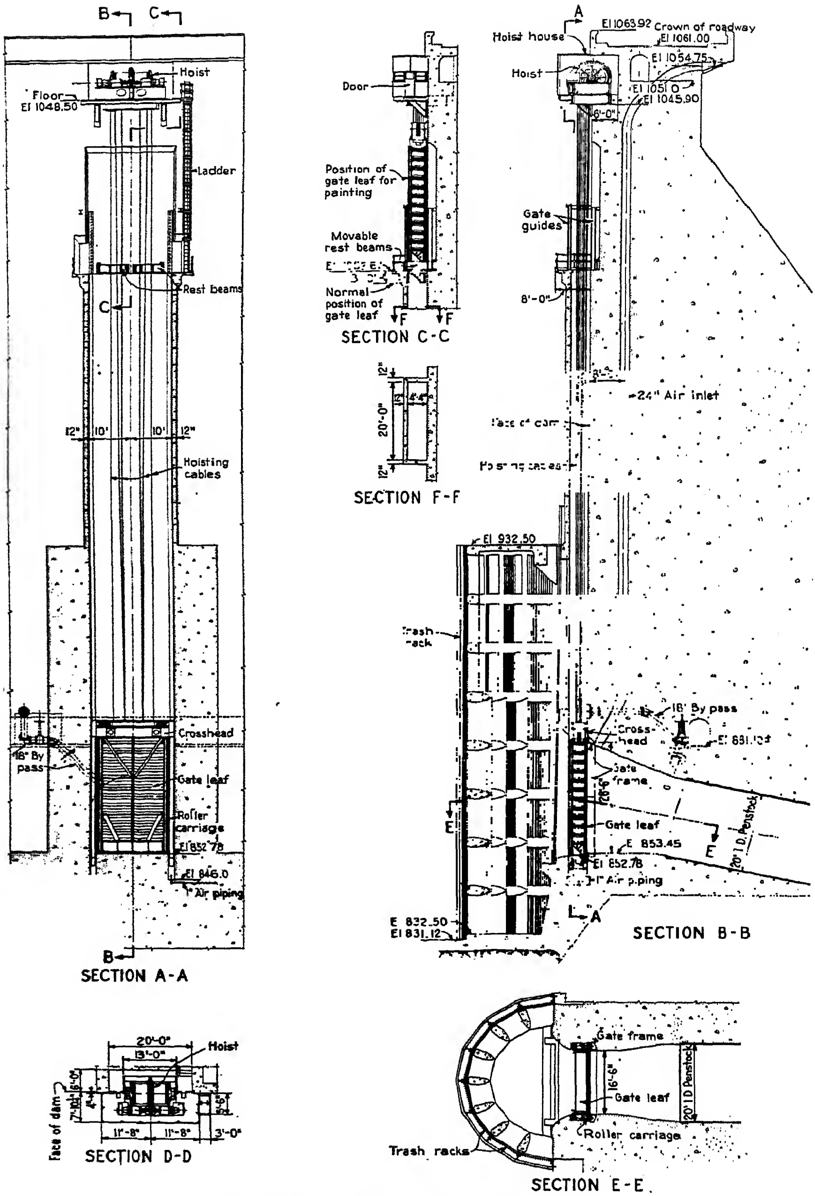

FIGURE 41.—*Penstock intake structure.*

not in use. When in use, they span the gate opening at the top of the guide structure.

Originally it was intended to streamline the beams and columns on the downstream side by means of attached steel plates. Later study revealed that the velocity was so low that extreme streamlining would have little, if any, effect on the efficiency of the intake. Also, there was a possibility that the steel plates might eventually come off and do serious damage. Consequently the plates were omitted, and the streamline effect was at least partially secured by suitably forming the concrete.

The trashracks are provided with compressed-air cleaning pipes arranged around the outside periphery of the structure at three locations. Additional outlets are provided in the tractor gate roller carriage pits. The outlet piping is 1½-inch standard brass perforated with ⅛-inch holes on 6-inch centers, alternate holes 15° and 30° with the vertical. The system is controlled from valves located in each gate hoist house.

The trashrack structure was designed to act as a closed half-cylinder and, assuming the flow of water to be obstructed by an accumulation of debris, to withstand a differential static head of water equal to 20 feet imposed on the roof slab and top ring and 40 feet on the remaining rings which support the rack bars.

The original design provided one trashrack, 120 feet wide and extending from elevation 831 to 1,012, for the two penstocks. This feature was redesigned to provide separate semicircular structures with top elevation at 932.5 for each penstock.

Tractor gates and hoists.

A tractor-type gate was installed at each turbine intake structure. The gates are designed to operate normally under a maximum hydrostatic pressure of 167 feet but are capable of prompt and safe closure under an emergency condition with an unbalanced pressure of 80 pounds per square inch against the leaves and a flow of 8,500 cubic feet per second through each intake. Normal operation is from the main push-button station located in each gate hoist house but they can be lowered in an emergency from a push-button station located in the control room of the powerhouse. The gates are interlocked so that they cannot be raised except when the penstock is filled with water, and during normal operation the water pressure will be at least partially equalized on both sides of the gate through an 18-inch bypass piping system.

Among the outstanding features of the gates is the complete elimination of sliding friction under heavy loads. This is accomplished by the use of rollers for both the vertical movement of the gate and the small horizontal upstream movement employed to separate the seals on the gates from the seats on the gate frames before the gate leaves are moved vertically.

Inclined, or wedge, roller trains are interposed between the sides of the gate leaf and its supporting roller carriages and are so arranged that the leaf, wedge roller trains, and the roller carriages are interlocked and move downward in unison when closing, until the leaf is opposite its penstock opening. Then the lower end of the leaf comes into contact with its stop in the gate frame and is restrained

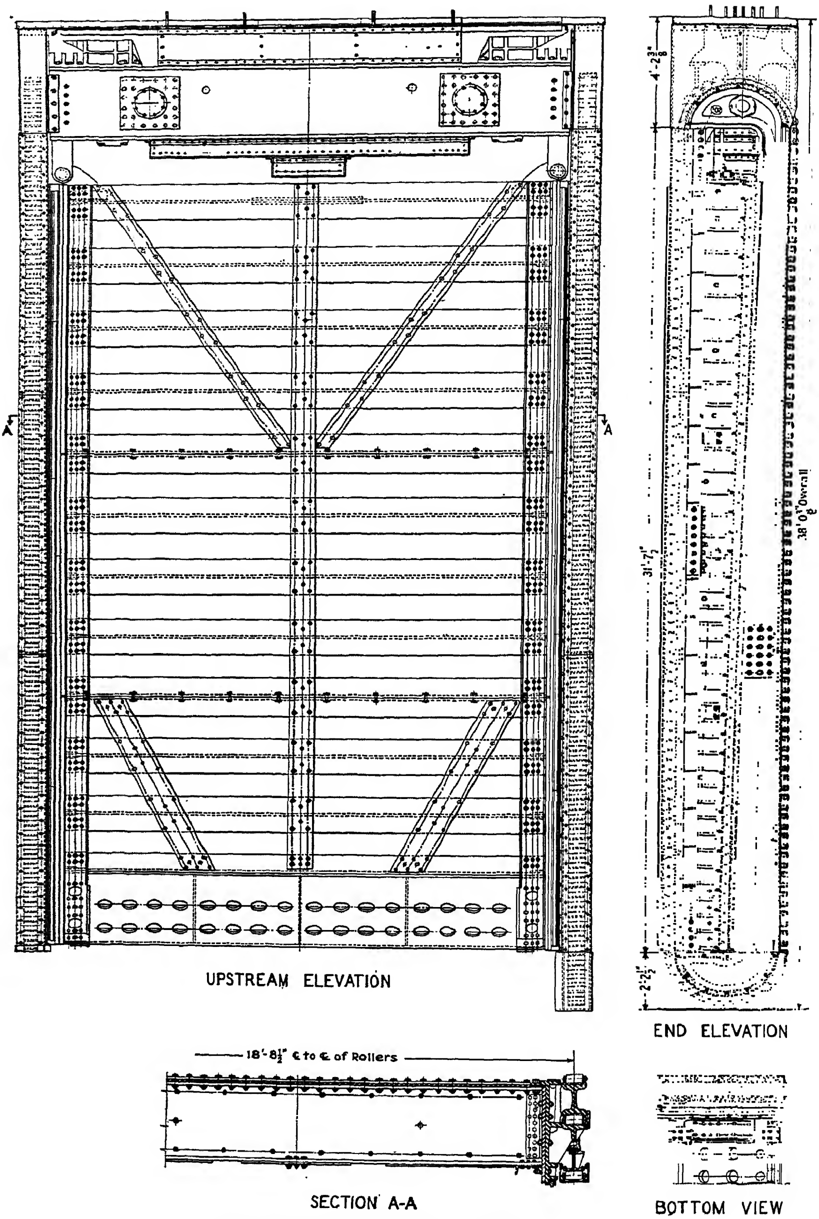

Figure 42.—*Details of tractor gate.*

from farther descent. The carriages continue their downward movement about 18 inches farther, and in so doing withdraw the support afforded by their wedge roller trains between the inclined surfaces on the leaf and the similarly inclined surfaces on its roller carriages, thus permitting the water pressure against the upstream face of the leaf to force it downstream approximately three-sixteenths of an inch. This is sufficient to bring the gate seals into watertight contact with the mating seats in the gate frame, and so complete the cycle of closure.

When beginning the opening cycle, the hoists first start raising the roller carriages which, in their ascent, bring the wedge roller trains into contact with the gate leaf and these inclined roller trains then act as practically frictionless wedges to force the leaf horizontally upstream about three-sixteenths inch away from contact with the gate frame seats. After this has been accomplished the lifting lugs in the lower portions of the ascending roller carriages engage the lower extremity of the leaf. Then the leaf, wedge roller trains, and roller carriages are again interlocked and move upward in unison.

Since no sliding of the sealing surfaces takes place, these surfaces cannot be damaged by the destructive action otherwise present when such sliding occurs, and hence they will remain permanently tight. An ingenious lever device is incorporated in the cross head to prevent the leaf from raising on the inclined wedge tracks, under the action of the water load during the seating operation.

A combination of structural and cast steel construction is used in the gates, the guides, and the frames. Some of the especially noteworthy features of design include the use of stainless steel for rollers and tracks, where both high strength and noncorrosive properties are desirable; the placing of the skin plate on the downstream side of the gate to eliminate flotation uplift when the gate is in the closed position; the reduction of downpull at partial openings by the use of a sloping, perforated bottom girder construction; the use of high-pressure air jets for cleaning the roller recesses; the installation of a special maintenance platform for periodic inspections; and the incorporation of an air vent for minimizing vibrations at partial openings under emergency conditions.

Each hoist is composed of two tandem aligned, mechanically connected, 60-horsepower, gear-reduction motors which drive a double drum by means of a ring gear which also joins the drums together, and the necessary controls and frame mountings for the motors, gear reducers, drums, and sheaves. Each hoist rests partly on sole plates embedded in the floor of a recess in the upstream face of the dam and partly on two triangular-shaped structural steel brackets which are supported by structural steel insets embedded flush with the face of the dam. The bolts provided in the top of the inset and completely embedded in concrete give the top of the inset additional anchorage.

Each hoist is connected to the gate which it handles by a 28-part, 1-inch diameter, 6- by 19-hemp core steel cable. The cable is in one piece and each end is permanently fastened to an outside drum flange. The hoist ropes were prestretched by a load of 18,000 pounds before shipping and were thoroughly impregnated with waterproofing and lubricating compounds.

The main limit switch for stopping the gate hoist is attached to the drum shaft through a geared driving mechanism which also drives

a selsyn transmitter. Each selsyn transmitter has a companion selsyn receiver on the main switchboard, which is connected to an indicator to show the position of the gate. The driving mechanism has a pointer attached which also indicates the position of the gate. The main limit switch has three sets of contactors which function during the operation of the gate as follows:

1. To lower the gate to the closed position the "lower" push button is pressed which starts the motors and lowers the gate. When the gate reaches the closed position, the hoist is automatically stopped by action of the relays which trip out the "lower" circuit when contact is made in the limit switch with the "lower" limit contactors.

2. To raise the gate from the closed position, the "raise" push button is pressed which starts the motor and raises the gate unless the circuit is kept open by the differential switch due to a low hydrostatic pressure in the penstock. When the gate reaches the normal raised position, the hoist is automatically stopped by action of the limit switch and relay which at "normal upper" position trip out the "raise" circuit. The normal raised position is about halfway from the penstock opening to resting beams. The gate cannot be raised unless the pressure differential switch is closed. This is actuated by two sylphons, one of which is connected to the reservoir through the net head piezometer line and the other to a scroll case piezometer line. It is closed only when the penstock is full of water and the difference in pressure upon the two sides of the gate is less than the permissible amount—the difference to which the differential switch is set.

3. A "maintenance" push-button circuit is provided through which the hoist is started when it is necessary to raise the gate above the "normal raised" position. Both the "maintenance" and "raise" push buttons have to be pressed and held in during such raising. The upward movement of the gate is stopped when it has reached the maintenance position by the second "upper" or "maintenance limit" switch and relay. The upward movement of the gate is stopped when being raised to the maintenance position by the second upper or "maintenance limit" relay contactor which is provided in the main limit device.

In addition to the main limit switch, three other limit switches are provided on each hoist to stop the hoist in case of failure of the main limit switch. One of the limit switches stops the hoist whenever the hoist cable becomes slack. The second limit switch is tripped by means of a rod which extends vertically below the hoist frame to an extreme top position of the gate. The third limit switch is attached to one of the driving motor shafts and stops the hoist when for any reason the normal operating speed of the motors is exceeded.

A thrustor-operated brake is provided on each hoist motor shaft and receives its energy from the same source as do the driving motors. While the motor is energized the brakes are off, and vice versa.

The main features of the hoist include the use of wound-rotor motors, permitting the acceleration of the load to full speed without excessive peaks of torque or current, at the same time retaining push-button control; regenerative braking when lowering under load; remote control from one station in the powerhouse for lowering only in case of emergency; and the use of oil-operated load brakes for smooth deceleration.

A hoist house was provided at each gate consisting of a structural steel frame covered with natural-color asbestos board and provided with one-piece wire glass windows mounted in steel frames. The framework is supported by the brackets which support the hoist and is anchored to the upstream face of the dam. The roof is of reinforced concrete slab construction poured in place.

The original design specified a butterfly valve in each of the two penstocks adjacent to the scroll case. Steel bulkhead gates were to have been used ahead of the penstocks. Economic and mechanical considerations favored roller tractor gates which were substituted for the butterfly valves and sliding steel bulkhead. The omission of the butterfly valves made it possible to place the powerhouse closer to the dam and to omit the gantry crane located over them.

Transition section.

The upstream part of each penstock, beginning at the upstream face of the dam and extending downstream to the steel penstock, consists of a concrete transition section from a rectangular opening 16 feet 6 inches wide by 28 feet 6 inches high at the upstream end to a circular opening 20 feet in diameter at the beginning of the steel lining.

Penstocks.

Two penstocks, each 20 feet in diameter, serve to conduct water from the reservoir to the turbines, one penstock serving each generating unit. Each penstock is lined with a plate steel pipe, 153 feet 10 inches long, having arc-welded longitudinal and circumferential joints. Beginning at the upstream end, the pipes slope downward in a downstream direction at an angle of 12°30′ with the horizontal for approximately 132 feet to an elbow in which the profile of the pipe is changed to the horizontal. The thickness of the plate decreases by sixteenths of an inch from 1⅛ inches for the upstream 30 feet of pipe to 1⅜ inches for the 34 feet of pipe at the downstream end. The pipes are solidly embedded in the concrete.

The penstock for the east unit is located on the center line of block 34 of the dam, and for the west unit is located on the center line of block 35. The upstream end of both pipes is 20 feet from the intersection of the center line of pipe with the upstream face of the dam. The center line of the horizontal section at the downstream end meets the center line of the scroll cases at elevation 832.

The steel pipe is designed to withstand the full internal pressure of the 215-foot head independently of the surrounding concrete. The pipe is made up in 10-foot lengths, with a 1 7/16- by 8-inch stiffener ring at the center of each 10-foot section and with three 1- by 6-inch seal rings, the first of which is located 6 inches from the upstream end, and successive rings spaced 6 inches on center.

The steel plates for the penstock pipes are of grade "B" steel,[11] and were made in accordance with Federal specifications.[12] All fabrication was in accordance with the Authority's specifications and with the requirements for unfired pressure vessels.[13] The welds were

[11] Standard A-78, A. S. T. M., Steel Plates of Structural Quality for Forged Welding.
[12] Federal Specification QQ-M-151, General Specifications for Metals.
[13] Rules for the Fusion Process of Welding, Class I, A. S. M. E. Boiler Construction Code, Unfired Pressure Vessels Section.

X-rayed and stress relieved as specified in the "Code" except that stress relieving of field joints was omitted after one field joint had been stress relieved.

The effect of stress relieving the circumferential joints by heating the weld and a limited area on each side of it while the remainder of the metal remained at air temperature was considered as possibly harmful. To determine the effect in an actual case, the first field joint of the west penstock was stress relieved and tests were made to determine the resulting state of stress. Ten-inch strain gage patterns of 20 lines each were laid out on the inside and the outside of the pipe, centered on the weld, and designed to measure longitudinal, circumferential, and diagonal strains. Initial measurements were made after stress relieving. The test piece was then removed by close drilling a 15-inch square. A comparison of measurements before and after removal of the test piece indicated the magnitude of the residual stresses. After removal the test piece was probably free of stress. The process was then repeated on a joint which had not been stress relieved. In the second case both 10-inch and 2-inch gages were used and each pattern consisted of 91 lines. A total of over 1,200 strain gage readings was taken. The measurements indicated that stress relieving of the steel joints introduced stresses approaching the ultimate stress of the steel. The test data and test results are included in chap. 6.

An 18-inch bypass line is provided connecting the reservoir with each penstock for the purpose of filling the penstock prior to opening the head gate. The flow of water through the bypass to each penstock is controlled by a manually operated 18-inch rising stem wedge gate valve located in a recess in the upstream side of the operating gallery (see fig. 41).

A 24-inch air inlet is also provided for each penstock. The inlet terminates in the downstream side of the dam under the roadway parapet. The details of location and arrangement are shown on figure 41.

TURBINES

Two 66,000-horsepower, 112.5 revolutions per minute vertical shaft, single runner, Francis type hydraulic turbines are direct connected to the generators. The turbines were designed and manufactured by Newport News Shipbuilding & Dry Dock Co.

The normal gross head on the turbines will vary between a minimum of 129 feet and a maximum of 207 feet. The tail water surface will vary between elevation 820.5 at no load and elevation 826.0 at full load. At extreme flood stage the tail water surface will reach elevation 860. The center line of the turbine distributors is at elevation 832, or 6 feet above the normal tail water surface..

The turbines are designed to withstand the stresses imposed by a runaway speed of 215 revolutions per minute, possible under the maximum available head of 207 feet with no load on the generator.

Runner.

The runner was made of cast steel in one piece with 19 blades, and has an over-all height of 6 feet 10 inches. The inlet diameter is 161 inches and the maximum diameter is 173.88 inches. Under rated conditions, the specific speed is 48.8.

The wearing ring of each runner is a continuous ring of rolled mild steel. The two outer wearing rings were shrunk on the runner. The inner one was pressed into its seat and secured by means of screws. The outer rings are opposite the crown plate and curb plate stationary

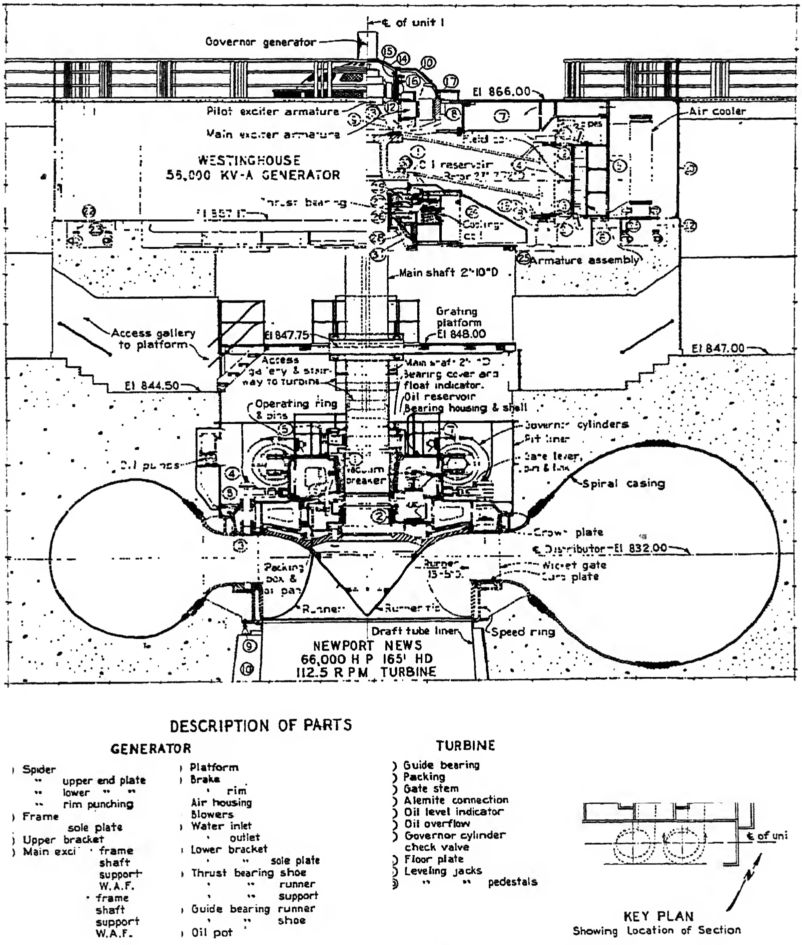

FIGURE 43.—*Section through generator and turbine.*

wearing rings, respectively, and the inner rings are opposite the inner crown plate seal ring.

To get the required close uniform running clearance between the rotating and stationary rings, the seats for the stationary rings were ground to final dimensions and concentricity during the field assembly.

The runner is secured to a flange on the lower end of the turbine shaft by means of fitted bolts and was designed to have a clearance between runner wearing rings and stationary wearing rings of 0.04 inch at the top and 0.06 inch at the bottom.

Speed ring.

The speed ring, of cast steel, was made in four sections with three vertical stays or columns in each section. It was designed to support the weight of the superimposed structures, including the weight of

FIGURE 44.—*Turbine runner.*

the generator and the scroll case (empty), and also to resist bursting stresses when subjected to an internal pressure of 125 pounds per square inch in the scroll case, but with no superimposed weight on top of the scroll case.

The bottom of the speed ring is bolted to the top ring of the draft tube, the pit liner is bolted to the top of the speed ring, and the scroll case is riveted to the speed ring. The entire assembly was concreted into the powerhouse substructure.

Scroll case.

The scroll case was made of steel plate varying in thickness from 1$^{11}/_{16}$ inches to 1 inch. It is substantially circular in cross section

and of such size that the velocity of water does not exceed 20 percent of spouting velocity under a head of 180 feet, or 21.5 feet per second. The casing is 17 feet 8 inches in diameter at the inlet end and is connected to the penstock by a tapered thimble.

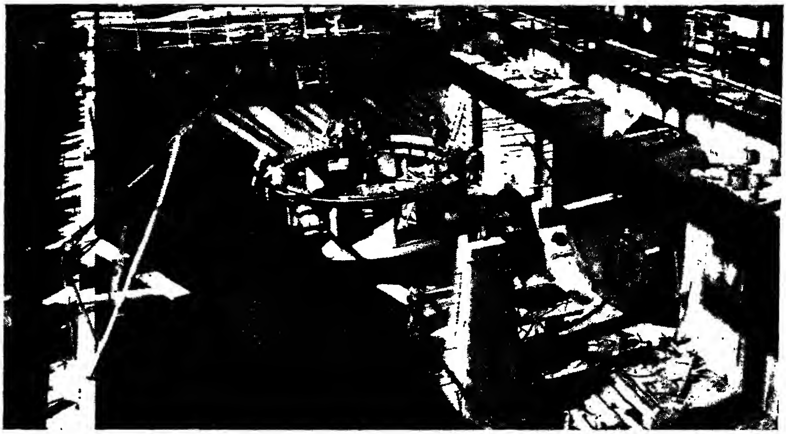

FIGURE 45.—*Speed ring.*

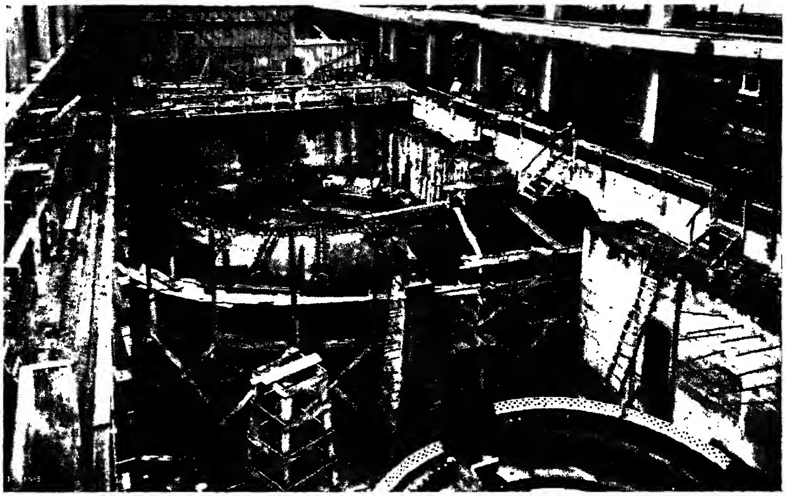

FIGURE 46.—*Scroll case.*

Steel plates forming the scroll case were formed, drilled, erected, and matchedmarked in the shop. Forty thousand rivets were used for the two casings.

A 24- by 36-inch manhole is provided in each casing. In order to prevent accidental opening before the casing is unwatered, each manhole door has a hinged cover which opens into the scroll case and is provided with backing-out screws.

Considerable discussion ensued during the course of acceptance of the scroll case design concerning the part of the casings between section 10 and section 18 where the stresses [14] were high. As a result, the cross-sectional shape of the casing in the small end was

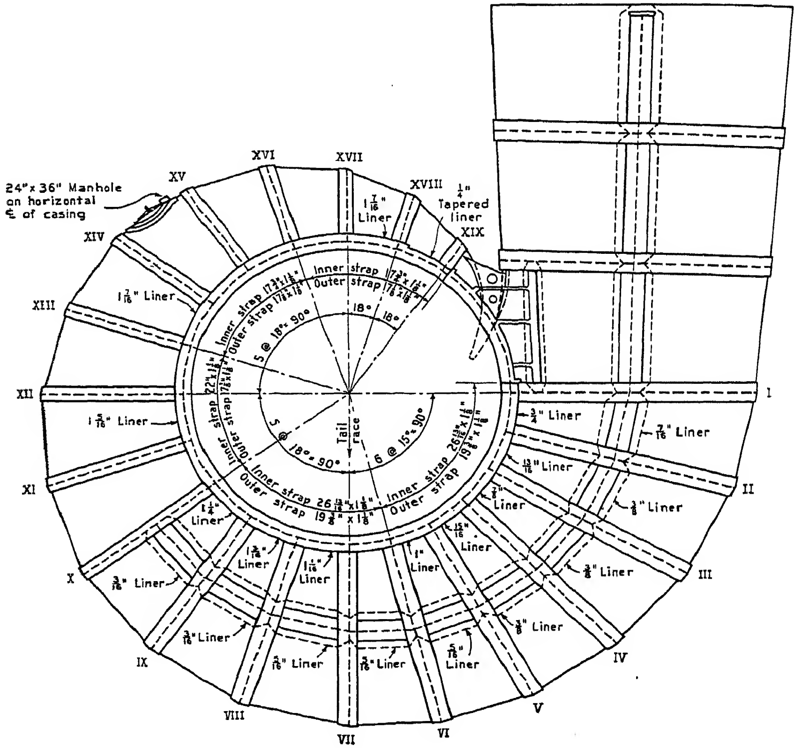

FIGURE 47.—*Details of scroll case.*

redesigned to effect a decrease in the radius of curvature on the horizontal center line of the distributor. Such modification resulted in an increased curvature along the girth at the outside of the case and brought about an equalization of the longitudinal and girth stresses in this region. A comparison of the original stresses (in parentheses) and the revised stresses as calculated is shown in table 24.

[14] Band, Zewiter; Auflage, Zewiter; Foppl, Von A.; and Foppl, L., Drang and Zwang, art. 58, p. 7. Timoshenko, S., Strength of Materials, vol. II, art. 27, p. 510, and art. 28, p. 515.

TABLE 24.—*Comparison of original and revised stresses*

Section number	Plates at point 3 longitudinal stress [1]	Girth joints: Shear in rivets [1]	Girth joints: Tension net section [1]	Section number	Plates at point 3 longitudinal stress [1]	Girth joints: Shear in rivets [1]	Girth joints: Tension net section [1]
X	5,180 (6,840)	7,180 (9,480)	8,000 (10,520)	XV	7,620 (13,330)	10,550 (18,480)	11,750 (20,500)
XI	5,280 (6,540)	7,300 (10,290)	7,280 (11,300)	XVI	8,450 (15,060)	11,700 (20,900)	13,000 (23,180)
XII	4,530 (8,710)	6,270 (13,560)	7,000 (15,060)	XVII	10,500 (16,090)	14,610 (22,300)	16,270 (24,800)
XIII	5,620 (10,900)	7,770 (15,080)	8,660 (16,750)	XVIII	14,750	20,450	22,800
XIV	6,170 (11,890)	8,550 (16,500)	9,520 (18,300)				

[1] Pounds per square inch. Original stresses in parentheses.

These figures are based on the premise that all stresses are taken by the plate steel casing and none transferred to the concrete. The manufacturer's usual assumptions are that the concrete carries an outward load due to the difference in major and minor axes of the oval at the small end of the casing. In figuring girth stresses each section of plate was considered independently as a circular section, the diameter of which is the average of the major and minor axes of the oval. In figuring longitudinal stresses the load was taken as the projected area of the section, the same as for a cylinder head. By this method of analysis the stresses were as shown in table 25.

TABLE 25.—*Girth and longitudinal stresses*

Section number	Girth stress point 3 [1]	Longitudinal stress at joint [1]	Section number	Girth stress point 3 [1]	Longitudinal stress at joint [1]
X	3,280	6,900	XV	3,130	6,350
XI	3,360	7,040	XVI	2,900	5,770
XII	3,220	6,730	XVII	2,700	5,250
XIII	3,500	7,210	XVIII	2,400	4,360
XIV	3,340	6,850			

[1] Pounds per square inch.

The longitudinal stresses at a joint corresponding to section XVII was computed for several installations upon the assumption that all stresses were carried by the plate steel. As noted from table 26 the casing as revised was relatively the strongest ever built by the manufacturer.

TABLE 26.—*Comparison of various scroll cases*

	Rochester No. 5	Deer Lake New Foundland	Dnieprostroy	Shoshone	Norris
Inlet diameter-feet	12.1	9.5	25	5.5	20
Design pressure	65	139	80	200	125
Tension net section [1]	13,540	36,650	16,500	29,800	16,270
Based on joint efficiency	22,800	59,700	23,400	68,500	17,900

[1] Pounds per square inch.

The plate steel spiral casings were not built to the exact dimensions as designed. Table 27 shows the dimensions in feet to inside of plate both as designed and as actually built.

TABLE 27.—*Inside dimensions of plate as designed and as built*

Section	Horizontal			Vertical		
	Design	Unit 1	Unit 2	Design	Unit 1	Unit 2
I	27.479	27.542	27.625	16.5	16.479	16.417
II	27.042	27.104		16.125	16.063	
III	26.594	26.667	26.656	15.729	15.688	15.708
IV	26.135	26.229	26.229	15.333	15.208	15.271
V	25.656	25.760	25.729	14.917	14.792	14.917
VI	25.177	25.271	25.229	14.5	14.417	14.5
VII	24.667	24.813	24.729	14.063	13.917	14.0
VIII	24.094	24.313	24.25	13.583	13.292	13.354
IX	23.365			13.25	12.802	12.854
X	22.677	22.875	22.833	12.708	12.438	12.5
XI	22.0			12.208	12.146	12.167
XII	21.208	21.323	21.333	11.708	11.688	11.625
XIII	20.406	20.490	20.583	11.177	11.188	11.125
XIV	19.573			10.688	10.729	10.625
XV	18.557	18.625	18.677	10.063	9.917	9.854
XVI	17.521			9.563	9.438	9.438
XVII	16.417	16.417	16.417	9.042	8.917	8.958
XVIII	15.135	15.302	15.292		8.542	8.604
XIX	13.469			7.938	7.854	7.917

Wheel-pit liner.

The wheel-pit liner is made of ½-inch steel plate in two sections. It is 19 feet 6 inches in diameter inside and 7 feet high. The turbine

FIGURE 48.—*Wheel-pit liner.*

guide bearing oil pumps are situated in a recess in the liner. The liner is designed to transmit the weight of the servomotors to the mass concrete of the powerhouse substructure.

Curb and crown plates.

The curb plate is made of cast steel in two pieces and is bolted to the lower flange of the speed ring. It contains the lower bearings for the 24 gate stems. The bearings are lubricated with grease through the gate stems. The upper and inner faces of the curb plate adjacent to the gates and runner are provided with renewable steel wearing rings and plates. The wearing plates on both the curb and crown plates are made from rolled steel plates in four sections, with the ends fastened together to form a circle. The wearing plate on the curb ring is 1 foot $4\frac{7}{16}$ inches wide and varies in thickness from $\frac{15}{16}$ to 1 inch, while that on the crown plate is 1 foot $9\frac{11}{16}$ inches wide and varies from $\frac{60}{64}$ inch to $\frac{63}{64}$ inch in thickness.

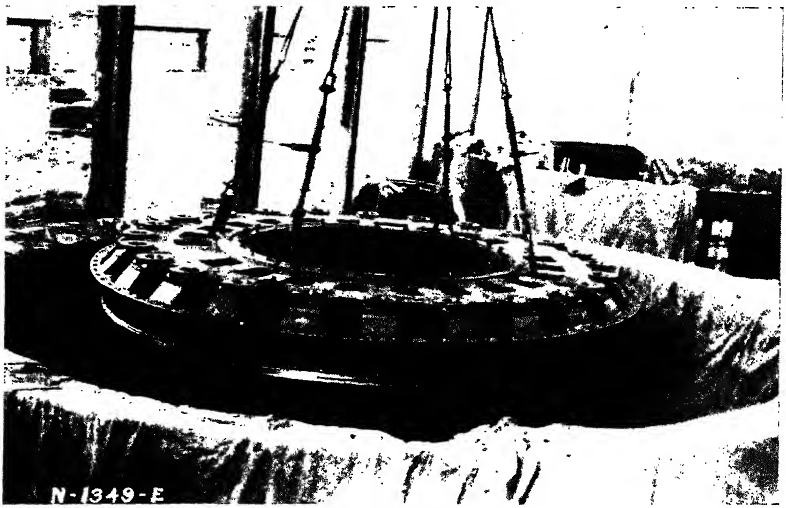

FIGURE 49.—*Crown plate.*

The crown plate is made of cast steel in two pieces and is bolted to the upper flange of the speed ring. It is reinforced with twenty-four $1\frac{1}{4}$-inch thick ribs and contains two bronze bushed bearings for each wicket gate stem.

An adjustable stuffing box equipped with $\frac{3}{4}$-inch-square asbestos packing is provided for each of the 24 gate stems. The weight of the gate and lever, together with any unbalanced hydraulic thrust, is carried on a thrust collar. The crown plate also supports the main-shaft bearing housing and oil pan.

Guide bearing.

Each unit has one guide bearing of the oil-lubricated, babbitted type. The bearing is 36 inches in diameter, with a contact surface 34 inches high, and consists of a cast-iron shell made in three sections and bolted to the bearing support. The cast-iron shells are lined

with babbitt grooved for oil circulation. The bearing cover is made of cast iron in two sections. The bearing support or housing is made of cast steel in three pieces and bolted to and supported by the crown plate.

Oil is circulated normally through the bearing by an alternating-current motor-driven geared oil pump located in a recess in the wheel-pit liner. The oil is stored in a covered sump on top of the crown plate and is surrounded by water which tends to cool the oil. Oil is pumped from the sump to a supply reservoir located in the bearing cover, from where it flows by gravity to and through the bearing

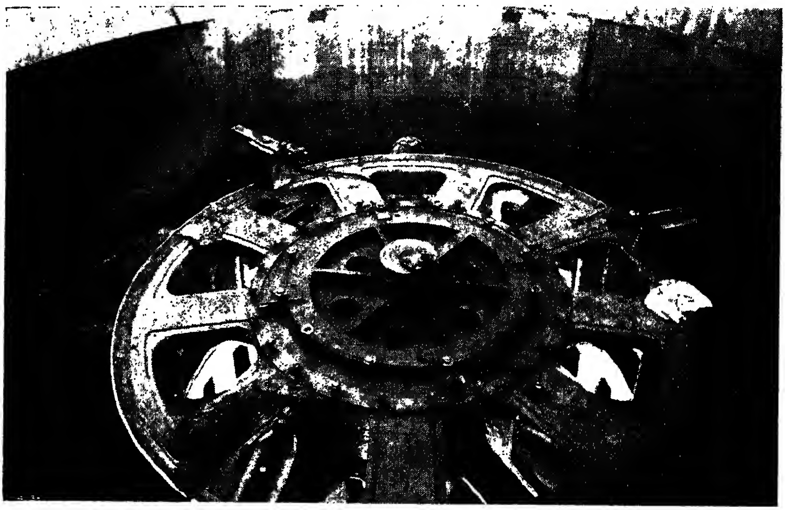

FIGURE 50.—*Guide bearing (grinding-mechanism housing in place).*

back to the sump. This pump is driven by a 460-volt, alternating-current, ½-horsepower motor. There is also an auxiliary pump in each unit driven by a 250-volt, direct-current motor from the station storage batteries. The auxiliary pump is arranged to operate when the alternating-current motor-driven pump should for any reason fail to deliver a sufficient amount of oil to the main bearing. An alarm is arranged to give a signal if the alternating-current supply should fail.

One temperature detector is located in each bearing segment where the temperatures are the hottest. Temperatures of bearing metal and oil are recorded by Leeds and Northrup Micromax recorders. The detectors are also connected to alarm contactors which set off an annunciator alarm when temperatures exceed a predetermined amount.

Shaft.

The shaft is made of forged carbon steel[15] and is 35 inches in diameter except where it passes through the bearing, where it is

[15] American Society for Testing Materials standards, A18–30, class E.

35.986 inches in diameter. Each end of the shaft is provided with a flange for connecting with the runner and the lower part of the generator shaft, respectively. The flanges are 5 feet in diameter and 7 inches thick and were forged integrally with the shaft. A 6-inch hole is provided through the center of the shaft for inspection purposes.

To secure proper alinement of the generator and turbine shafts, the turbine shaft was shipped to the generator manufacturer, where the two shafts were matched, the coupling bolts fitted, and the alinement checked.

A stainless-steel cover made in matched halves is provided to cover the coupling between the turbine and generator shafts. A renewable stainless-steel sleeve is fastened by shrink links to the shaft throughout the length of the stuffing box.

Wicket gate and operating mechanism.

The turbine gates are of the balanced wicket type made of cast steel and have integral cast stems. The gates are actuated by twin servomotors mounted in the wheel-pit liner. The reactions of the

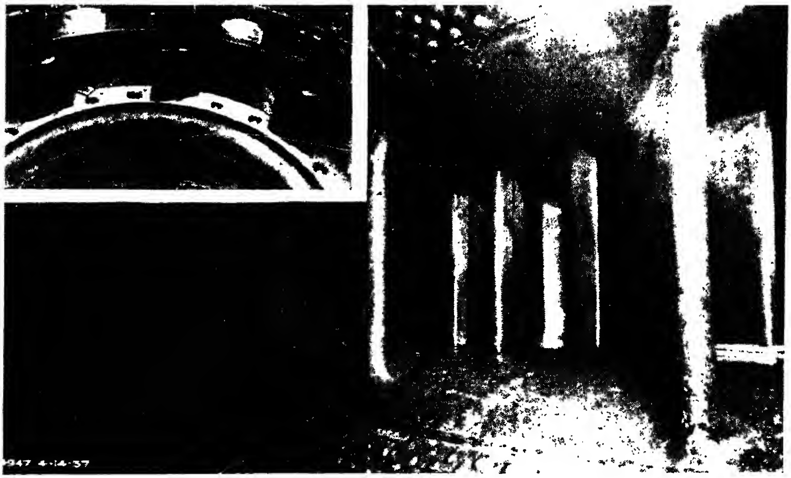

FIGURE 51.—*Wicket gate.*

servomotors are carried by foundation bolts which transmit the load to the concrete of the powerhouse substructure. The operating ring is supported by and operates around the guide bearing support.

The gate levers consist of three main parts, two of which have one end connected to the operating ring by means of an adjustable link, while the other end of these two pieces "floats" on the wicket gate stems. The other part of the lever, which is "sandwiched" between the first two mentioned pieces, when assembled has one end keyed to the wicket gate stem and the other end fastened to the other two pieces of the lever by a cast-iron shear pin designed to fail when the

normal operation of the wicket gate is interrupted by a foreign object caught in the gate, thus allowing the other gates to operate normally and prevent damage to other parts of the machine.

The wicket gates are held in suspension in midposition between the curb and crown plates by means of thrust bearings. Each gate is also provided with a steel thrust collar to carry any upward thrust due to unbalanced hydraulic pressure. The thrust collar bears against a flange provided on the upper bearing bushing of the crown plate. The collar is keyed to the gate stem and is provided with a lug to engage a corresponding lug on the crown plate to limit the angle of movement of the gate-stem lever in case of breakage of a breaking link. Interference will thereby be prevented between the loose gate and the other gates. The design is such that in case one gate comes loose from the shifting mechanism it will not swing to a position in contact with the runner.

The operating ring is made in one piece of cast steel and connected at opposite points on the periphery through 6½-inch diameter adjustable forged-steel connecting rods to the servomotors. The ring operates on bronze bearing rings which are attached to the guide-bearing support.

The servomotors are made of cast iron with cast-steel heads and stuffing boxes and are mounted in the walls of the pit liner on the upstream side of the pit. The load resulting from the operation of the servomotors is transmitted to the concrete of the powerhouse substructure by means of anchor bolts. The motors have cast-steel pistons with three cast iron piston rings. The servomotors are supplied with oil at 300 pounds per square inch pressure by the governor system. A device is provided on the servomotors to reduce the rate of closing of the gate after the position for speed-no-load has been reached in the closing direction. Provision is made on the servomotor connecting rods for locking the wicket gates in full-open or full-closed position or to limit the gate opening to maximum generator output when the turbine is operated under higher than rated heads.

The stress in the center of the yoke of the servomotor cylinder head at a section on the center line of the piston was calculated by the manufacturer to be 7,202 pounds per square inch.

Grease is supplied to these parts by a hose and hand gun from a 100-pound barrel-type compressor. A measuring control valve is provided in the grease line to allow a predetermined amount of grease to eject from the gun at each operation. All working joints and pin connections throughout the gate mechanism are bronze-bushed and provided with Alemite fittings for forced lubrication. Each gate stem bearing has a separate source of grease supply. Where the bearing is subjected to water pressure, a cut-off valve is provided on all grease connections between the bearing and the pressure grease fittings. A hole varying from 1⅜ inches to 1 inch was bored through the entire length of the gate stem to supply grease to the bearing in the curb plate. Although the grease system will normally work under pressures up to 1,250 pounds per square inch, a pressure of 2,500 pounds per square inch was applied to each gate to test for defects in the casting adjacent to the junction of the upper stem in the body of the gate, where the most severe stresses occur.

Draft tubes.

Water is conducted away from each turbine by means of an elbow-type draft tube. Beginning at the top and extending down 9 feet 2 inches, a welded plate steel liner is provided. The remainder of the tube is formed in the concrete of the powerhouse substructure. Two piers which start at the bottom of the elbow and extend to the

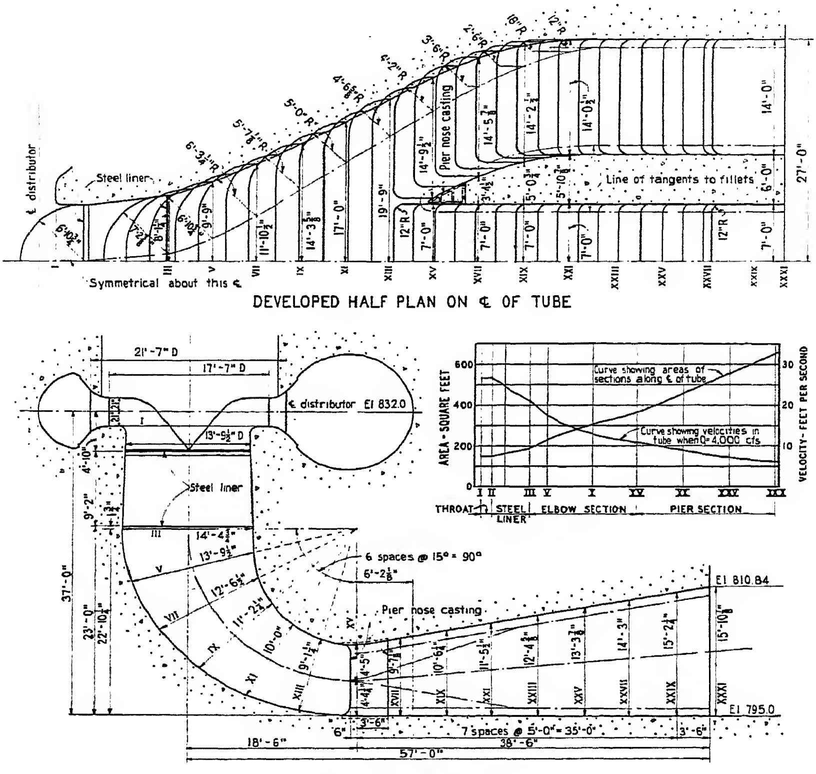

Figure 52.—*Details of draft tube.*

downstream end divide the tube into three discharge tubes. The floor of each draft tube is level, at elevation 795.

The original plans called for built-up steel and timber draft tube stop logs, metal lined stop log grooves and seats, and a steel gantry crane for lifting. Carefully constructed smooth grooves in the concrete were substituted for the grooves with metal liners and seats. Since only light timber logs are to be used, a gantry, as originally planned, was not needed and a simple wooden "A" frame is to be used on the infrequent occasions when it becomes necessary to use stop logs.

Immediately downstream from the discharge end of the draft tubes, the tailrace is paved with concrete for about 25 feet. The paving starts on a curve of 55-foot radius which intersects a 5 : 1 upslope, extending downstream to intersect the neat line of the tailrace at elevation 817. A 24- by 36-inch manhole, with hinged cover and backing-out screws, is provided through the draft tube liner for access to the interior of the draft tube.

Vacuum breaker.

An 8-inch vacuum breaker is provided at the crown plate. The breaker is actuated by the operating ring and is adjustable to open at a predetermined gate opening for the purpose of admitting air to the under side of the runner. This breaks the vacuum in the draft tube upon gate closure and is expected to improve the operation of the turbine at low gate opening. A pipe from each vacuum breaker is terminated in the roof of the powerhouse to minimize the suction noise.

Piezometer lines and turbine flowmeters.

Piezometer lines are provided to facilitate turbine testing and to secure a continuous record of flow through each turbine. Four piezometer taps are provided at each of two transverse planes on the penstock to facilitate turbine tests by the Gibson method. Each group of four is arranged around the circumference, each tap being placed 45° from the vertical and horizontal center lines. The first group is 37 feet from the upstream face of the dam, and the second group 91 feet 10 inches downstream from the first. Six taps are provided for the turbine flowmeters located in a transverse plane around the circumference of the scroll case, 37°30′ from the downstream section of the transverse center line of each unit, measured in a counterclockwise direction. Four additional taps, located 34 feet 9 inches upstream from the longitudinal center line of the turbine, are provided for the indication of net head. In addition, a pressure header pipe terminates in each hydraulic cubicle. A line from elevation 902 inside each trashrack terminates in the common valve box for each unit in the downstream face of the dam.

Each unit is provided with a Bailey flowmeter for recording the total discharge and also indicating the discharge of each turbine. The meters operate on the Winter-Kennedy principle and are designed to register flows between 5,000 cubic feet per second and 500 cubic feet per second correctly within 2 percent.

Acceptance test.

The acceptance tests [16] to determine the efficiencies of the two turbines were conducted during the period of October 20–25, 1937. The tests were made according to the Gibson method[17] for determining the efficiency of hydraulic turbines, and the Gibson time-pressure method was used for determining the quantity of water. Engineers employed by Norman R. Gibson, consulting engineer, with the aid of some

[16] Gibson, Norman R., Efficiency Test by Gibson Method, units Nos. 1 and 2, October 20–25, 1937. December 30, 1937.

[17] Gibson, Norman R., Gibson Method for Field Testing of Hydraulic Turbines, Mechanical Engineering, vol. 52 (1930), p. 374.

personnel recruited from the Authority's construction and operating organizations, conducted the tests. A representative of the Newport News Shipbuilding & Dry Dock Co. and representatives of the Authority observed the tests. The results are summarized in table 28.

TABLE 28.—*Summary of acceptance test results*

Unit No.	Net head-feet	Turbine efficiency at 60,000 horsepower		Turbine efficiency at 75,200 horsepower	
		Guaranteed	Actual	Guaranteed	Actual
		Percent	*Percent*	*Percent*	*Percent*
1	180	91.0	93.1	86.0	88.9
2	180	91.0	93.3	86.0	89.9

Unit No.	Net head-feet	Maximum turbine efficiency occurs at—			Maximum capacity occurs at—		
		Generator	Turbine	Discharge	Generator	Turbine	Discharge
		Kilowatts	*Horsepower*	*Cubic feet per second*	*Kilowatts*	*Horsepower*	*Cubic feet per second*
1	180	45,400	62,000	3,259	57,400	78,300	4,560
2	180	42,400–45,400	58,000–62,000	3,045–3,255	57,400	78,300	4,562

A detailed discussion of the acceptance test is given in appendix F.

GOVERNORS

The two Woodward governors are of the oil-pressure, relay-valve actuator type with electrically-driven, speed-responsive elements. The actuator and oil pump for each governor are mounted on the sump tank and are enclosed in cabinets made from welded plates.

Inspection doors at the sides of the cabinets allow access to the actuators and pumping equipment. Each cabinet is made of five pieces to facilitate removal. The arrangement of pumps, tanks, and controls is such that operation can be either as two independent unit systems or as one twin system. Each governor is complete with actuator, sump tank, oil pump, interconnecting oil piping, oil pipe to servomotors, permanent magnet generator which drives the speed-responsive element, and all necessary parts and accessories required by the governor system for regulating speed by controlling the turbine wicket gate.

Servomotors.

The servomotors are double-acting and are rated at 300,000 foot-pounds with a net oil pressure of 200 pounds per square inch in the cylinders and a pressure of 250–300 pounds per square inch in the system. The volume of oil per stroke is 16,900 cubic inches, and the ports of the servomotors' cylinders are 5 inches in diameter. They are provided with adjustable bypass connections so that the rate of closure may be retarded during gate travel from slightly below speed-no-load position to the fully closed position. The mechanical connection between the governor restoring mechanism and the turbine gate operating mechanism consists of ¼-inch diameter airplane type flexible cable enclosed in 1-inch pipe and connected to the piston

rod of the servomotors. The cable is supported on grease-packed ball-bearing sheaves. Weights on the actuator end compensate for any lost motion.

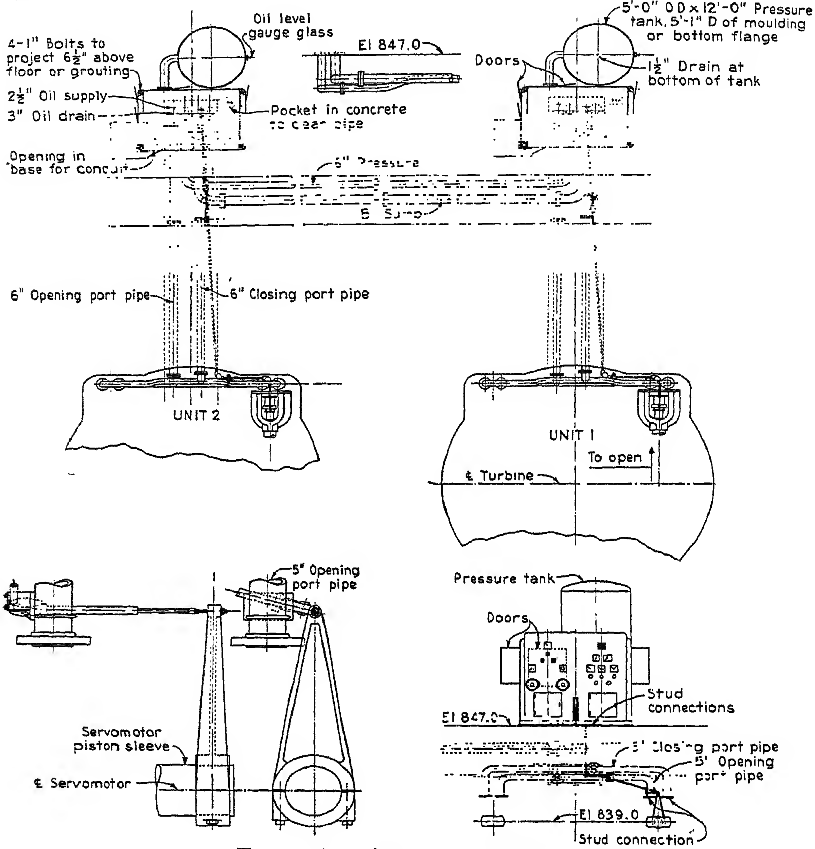

FIGURE 53.—*Governor installation.*

Actuators and controls.

The actuators are designed with adjustments so that the servomotors will move the turbine gate over the full opening or full closing stroke in from 4 to 10 seconds. The adjustment positively restricts the oil flow to permit a maximum rate of gate closure of 4 seconds and is arranged so that the operation of the speeder mechanism, control solenoids, or other devices cannot cause the rate of movement of the turbine gate to exceed the maximum rate for which it is adjusted. On the closing cycle provisions are made for cushioning up to 5 seconds from 0.1 to 0 gate.

The speed-responsive element of each actuator is driven by an alternating-current motor which receives its power from a permanent

magnet generator mounted on the main generator shaft. It supplies current exclusively for driving the governor head of the actuator. Included as a part of the assembly are an electric tachometer magneto and an overspeed switch.

The actuator relay valve passes oil to effect a corrective movement of the turbine gate upon a speed variation of the turbine of $\frac{1}{100}$ of 1 percent or more. Speed of the responsive element varies directly with and in exact proportion to the speed of the main shaft of the turbine for all rates of acceleration and deceleration. The servomotor oil control mechanism is capable of going from no-flow to full-flow position or vice versa in $\frac{1}{10}$ second, starting from the time of speed change.

Each actuator is provided with the following equipment:

1. *Load limit.*—Gate limit control device which can be operated manually at the actuator and also electrically by split-field direct-current motor from the switchboard.

2. *Speed level.*—A speed level control device which is operated manually at the actuator and also electrically from the switchboard by means of a split-field direct-current motor. The speed control has a range from 85 percent rated normal speed, no-load, and 0 speed droop to 105 percent rated normal speed at rated load and maximum speed droop.

3. *Speed droop.*—A device for controlling the speed droop of the turbine is operated manually at the actuator and electrically from the main control board by means of a selsyn transmitter and receiver. The amount of speed droop is adjustable from 0 to 6 percent.

4. *Automatic shut-down.*—A latching device for closing the turbine gate at the normal rate of closure, which is operated manually at the actuator and also electrically from the switchboard. The electrical control of this device is arranged so that it can be reset manually at the actuator or electrically at the switchboard. This device will be used for shutting down the turbine automatically upon overspeed, failure of governor oil pressure, failure of the oil supply to the main shaft bearings, or upon operation of any of the automatic protective features in connection with the main generator, exciter, and pilot exciter. The solenoid coils of the electrical devices are designed for continuous service at 250 volts direct current.

5. *Speed indicators.*—Two electrically operated speed indicators are provided which not only indicate the normal speed of the turbine but also show when the machine begins to rotate and when it comes to a dead stop. One indicator is mounted on the actuator panel and the other on the main control switchboard. A magneto-type generator connected to the generator shaft is provided for operating the speed indicators.

6. *Manual control.*—The turbine gates can be controlled by means of oil pressure from the governor oil pressure system by manual control at the actuator.

7. *Pressure gages.*—A duplex pressure gage mounted on the actuator indicates the air pressure in the station air system and also in the generator brake cylinders. Another pressure gage indicates the pressure in the governor oil-pressure tank.

8. *Limit and position indicators.*—A selsyn type gate-limit and gate-position indicator of the dual type is provided and mounted

on the main control switchboard and a similar indicator is mounted on the actuator panel.

9. *Overspeed switch.*—An overspeed switch is mounted on the generator shaft to shut down the turbine, trip the generator oil circuit breaker, and sound an alarm for overspeed. The overspeed contact is adjustable to operate at any speed from 140 revolutions per minute, or 125 percent of normal, to speed of 170 revolutions per minute. On the loss of the load with full gate opening, the turbine speed should not reach the maximum setting of the overspeed switch when the governor control is functioning normally. The contacts of the overspeed switch are automatically reset when turbine speed returns to 105 percent of normal speed.

10. *Hand-operated brake valve.*—A hand-operated air valve for controlling the operation of the generator brakes is provided on the actuator panel. The valve is provided with interlocks so that the brakes cannot be applied until the turbine gates are fully closed and the generator disconnected from the system.

11. *Automatic brake valve.*—This valve for controlling the operation of the generator brakes is mounted on the actuator panel. The valve control is arranged so that the brakes cannot be applied until the turbine gates are fully closed and the generator disconnected from the system. A time-delay feature is provided to prevent the brake application until speed of generating unit has decreased to about 50 percent of normal. Brake application is intermittent, and the time periods are adjustable.

12. *Two oil pressure switches.*—One of the switches initiates an alarm when the governor oil pressure drops to a predetermined and adjustable value. The other switch closes on extremely low oil pressure and causes the unit to shut down if running and prevents it from starting if at a standstill.

Oil-pressure system.

Each governor oil system includes a motor-driven pump, pressure tank, sump tank, and piping and accessories for supplying oil at the required pressure to the governor. The normal operating pressure is between 250 and 300 pounds per square inch, and the entire system is designed for an operating pressure of 300 pounds per square inch.

The motor-driven oil pump for each governor is of the herringbone type, with a capacity of 219 gallons per minute at 300 pounds per square inch pressure and is driven by a 60-horsepower, 440-volt squirrelcage motor. It is equipped with a check valve, oil pilot operated unloader valve, pressure safety valve, and pressure control switch. The pressure switch mechanically operated from the unloader valve assures that the pump motor will be started and stopped without load. The pressure safety valve insures against excessive pressure.

An automatic control is provided which regulates the oil level in the pressure tank. When operating as a twin system, the interconnection and automatic control of the two pumping units are such that either may be used for normal operation, with the other serving as a stand-by. The unit serving as a stand-by is arranged to start automatically, either on failure of the electrical supply to the operating unit or when oil pressure or level falls a predeter-

mined amount below the pressure or level which starts the supply of oil from the operating pump. The stand-by pump then delivers oil together with the operating pump until manually stopped. An alarm is sounded when the stand-by starts.

Each pressure tank has a capacity of 196 cubic feet, which is equal to 20 times the servomotor volume. Each tank is of all-welded steel construction and is provided wth a manhole, a sight gage to indicate oil level, and a pressure or liquid level regulator for automatically maintaining the proper quantity of oil and pressure in the tank. An air blow-off valve is provided on each tank which automatically discharges the air if pressure exceeds 325 pounds per square inch. Under normal operation, the tank contains approximately one volume of oil to three volumes of air at 200 pounds. All connections to the pressure tank except the air and the upper gage connection are made below the low oil level.

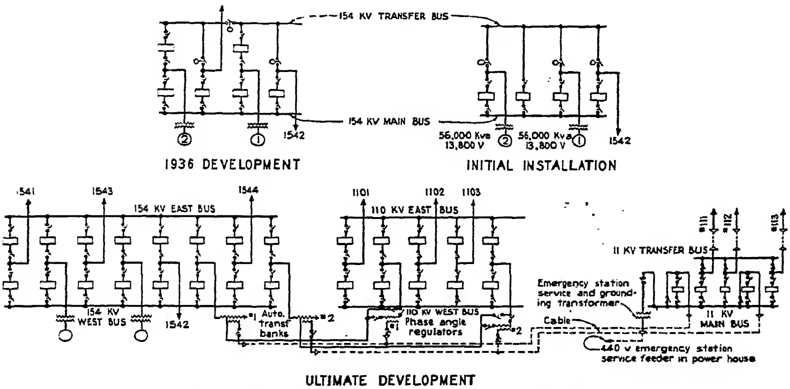

FIGURE 54.—*Single-line development diagram.*

The two 120-cubic-foot sump tanks for the two governors are of welded boiler plate construction. They are interconnected so that either tank may be drained for cleaning while the other is in service. Each tank is provided with a strainer through which all oil returned from the servomotors is passed. The sump tanks are each provided with a float-operated oil-level gage for indicating the quantity of oil in the tank. The sump for each governor is mounted in the base of the actuator with pumps above.

All the connecting piping between the oil pumps, sump tanks, pressure tanks, and actuators, and between the actuators and the servomotors is of seamless steel tubing, with long radius bends and welded wherever practicable. The pipe is of such size that the oil velocity for full gate travel in minimum specified time does not exceed 12 feet per second.

GENERAL PLAN—ELECTRICAL EQUIPMENT

The initial installation provided for two generators with transformers stepping up to the 154-kilovolt buses and one 154-kilovolt line to Wheeler Dam. The well-known main-and-transfer-bus scheme

of connections was provided with one spare breaker. Structures were so arranged that any of the motor-operated disconnects on the transfer bus could be replaced at any time by breakers, thus gradually expanding the scheme into a full double-bus-double-breaker scheme.

The 1936 extension consisted of adding a line to Alcoa and also adding a second breaker in each generator circuit and a motor-operated disconnect in the transfer bus. With these added facilities, two spare breakers are available and each generator may be operated on a separate line, isolated from the other if desired. Space is available in the switchyard, the conduit system, and the switchboard room for the further extension of the 154-kilovolt yard and the addition of 110-kilovolt and 11-kilovolt yards as may be required.

GENERATORS

The two generators were designed and manufactured by Westinghouse Electric & Manufacturing Co. Each has a normal rating of

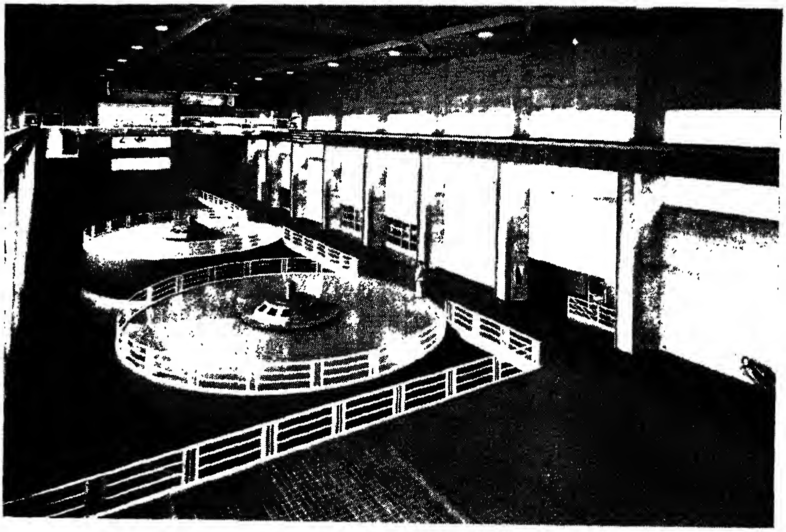

FIGURE 55.—*Generators.*

56,000 kilovolt amperes, 13,800 volt, 0.9 power factor, and a speed of 112.5 revolutions per minute. Each generator is provided with direct-connected exciter and pilot exciters, thrust and guide bearings, an enclosed ventilating system, surface coolers, and carbon dioxide protection.

Stator.

The stator frame is of welded plate steel construction. Openings are provided in the frame for the passage of cooling air. The frame, core, and windings were shop-assembled in four sections to facilitate shipment.

The stator core is built up of high-grade, nonaging silicon steel laminations, coated on both sides with insulating varnish, and is keyed and clamped to the stator frame. Ventilating spacers are provided in the vent ducts between sections of the core so as to provide for quiet and efficient flow of cooling air.

The stator winding is star connected. Each phase is wound in two parallel paths with six leads at the neutral end and three at the main end to accommodate current transformers for differential and split conductor relay protection. The neutral current transformers and the neutral connection are within the generator housing. The stator winding is insulated with class B insulation. The stator coils consist of several conductors, each of which is subdivided

FIGURE 56.—*Generator stator.*

into strands, transposed between coil connections to give equal current distribution between strands. The strand conductor and the completed coil insulation are mica taped. During the process of manufacture, each coil was impregnated several times and steam-pressed while hot so as to obtain a compact, homogeneous coil free from air pockets. An outer protective layer of asbestos tape is provided on the portion of the coil embedded in the core, and this is covered with a semiconducting graphite solution which provides corona shielding.

Twelve 10-ohm resistance temperature detector coils are distributed in the stator windings to detect the highest operating temperatures. The six coils giving the highest readings are connected to a temperature recorder. Three wires are brought out from each detector to a terminal block on the generator frame where one wire from each detector is grounded, and all three are extended to the switchboard.

The stator rests on foundation sole plates to which it is dowelled and bolted. Radial jack screws are provided on the sole plates for obtaining uniform air gap.

Rotor.

The rotor spider is constructed of cast steel with a solid hub. The rim is of laminated steel, clamped by through bolts between heavy top and bottom steel plates, thus constituting a solid, self-supporting, floating rim keyed to the ends of the spider arms in such manner as to transmit only tangential forces. The spider structure is thus relieved of the centrifugal stresses from the rim. The field poles are built up of thin steel laminations riveted together under high pressure. Dovetail projections on the poles engage with dovetail slots in the rotor rim, and the poles are held firm by tapered keys and wedges. They can be removed if necessary without removing the rotor from the stator.

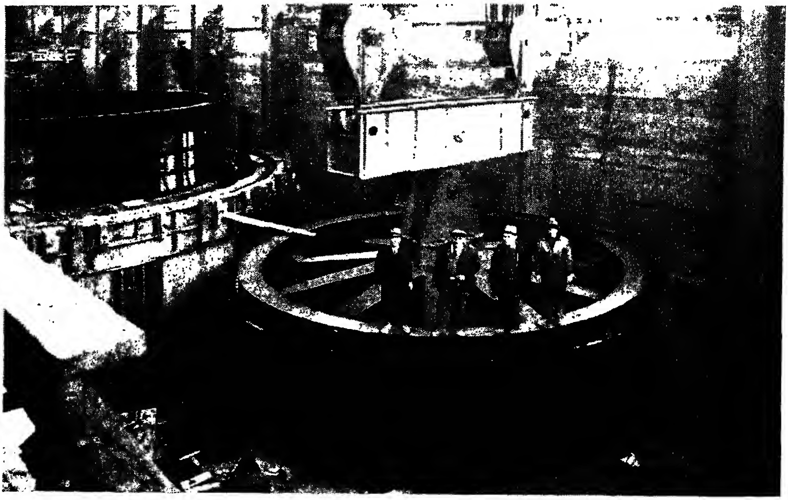

FIGURE 57.—*Generator rotor.*

The field coils are of copper straps wound on edge. Each coil is insulated from its pole by built-up mica and asbestos material. Duck micarta insulating collars are provided at the top and bottom of each field coil.

The rotor of each generator is provided with a segmental brake ring with which the brake shoes make contact. This segmental brake ring is removable and renewable.

Generator shaft.

The generator shaft is made of forged steel, having a flange at the lower end of the proper dimensions for coupling to the turbine shaft and another flange at the top of the shaft which acts as the thrust-bearing collar and to which the generator rotor is bolted and keyed.

The shaft is 34 inches in diameter and accurately machined throughout. A hole 6 inches in diametor was bored throughout the entire length of the shaft and machine-finished sufficiently smooth to permit visual inspection of the interior of the shaft.

Bearing brackets.

The upper bracket which supports the stationary parts of the exciters consists of eight fabricated steel arms bolted together. The outer ends of the bracket arms are supported by pads on the top of the stator frame.

The lower bracket which supports the combined thrust and guide bearing consists of eight fabricated steel arms bolted and keyed to the fabricated steel thrust bearing pot. The outer ends of the bracket arms span the wheel pit and are supported on the lower bracket base plates. The lower bracket was designed for removal through the stator bore and is provided with adjusting screws for centering.

Provision is made in the bearing and bracket to prevent stray electric currents from passing through the thrust and guide bearing.

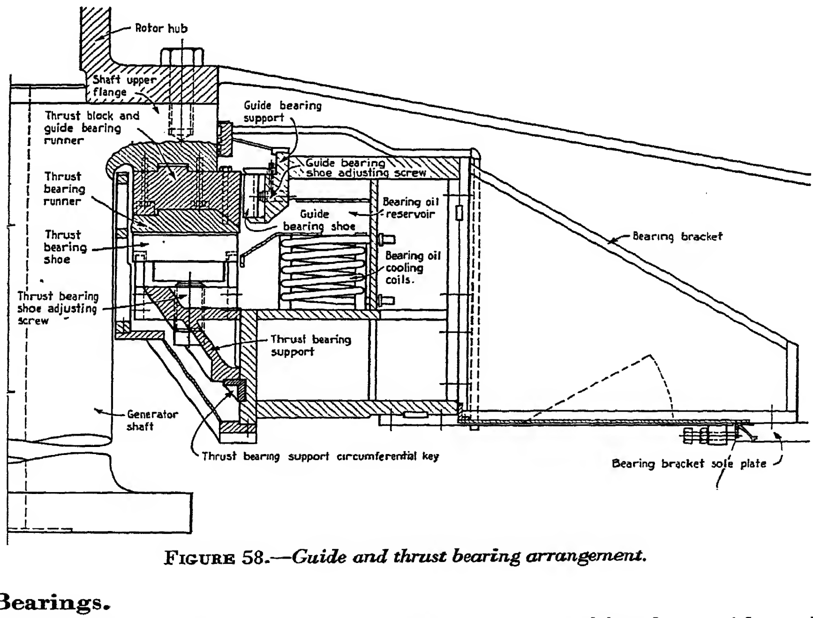

FIGURE 58.—*Guide and thrust bearing arrangement.*

Bearings.

Immediately below the rotor is located the combination guide and thrust bearing. The guide bearing consists of a number of segmental shoes individually pivoted and adjustable and bearing against the guide bearing runner which is immediately above and equal in diameter to the thrust bearing runner plate.

The thrust bearing is of the Kingsbury type and is designed to support the entire weight of the rotating parts of the generator and

water wheel and the hydraulic thrust. For lubrication, the thrust bearing is fully submerged and the guide bearing partially submerged in oil. The oil reservoir is a complete, self-contained, lubricating system when filled and requires no pumping. The oil is cooled by water circulating through copper coils, and suitable devices are provided to prevent throwing of oil and the escape of oil vapors. The oil level device for the reservoir was designed so as to sound an alarm if the oil level raises ½ inch above normal running oil level or drops ¾ inch below the stationary cold oil level.

Both the guide and thrust bearings have resistance-type temperature detectors in the bearing shoes for recording temperatures at the hydraulic gage boards.

The thrust bearing is 74 inches in diameter, about 2,750 square inches in area, and supports a rotating weight which produces a pressure of about 400 pounds per square inch.

Brakes.

Each generator is provided with air-operated brakes of sufficient capacity to bring the rotating parts of the generator and the turbine to a stop from half-rated speed within 7½ minutes after the brakes are applied. This can be done without injurious heating of the brake shoes or braking surfaces on the generator rotor, without field excitation on the generator, and with leakage through the turbine gates not exceeding an amount which will produce 1 percent of the rated torque. The brakes are electrically controlled and are designed for operation with a pressure of 80 pounds per square inch. The installation consists of 16 cylinders with pistons and brake shoes which are applied to the brake ring mounted on the under side of the rotor.

The brakes are designed to serve as hydraulic jacks to lift the generator rotor, shaft, and water wheel runner for removal or adjustment of the thrust bearing. The lifting jacks are capable of lifting the rotor and shaft at least 1½ inches. When the jacks are in the raised position, provisions are made for blocking the rotor mechanically in this position. A portable high-pressure oil pump which supplies oil at 1,500 pounds pressure is used when operating the brakes as lifting jacks. The supply piping is arranged to drain the oil and prevent it from entering the station air-piping system.

The brake shoes are provided with renewable metallic asbestos friction wearing surfaces.

Ventilation.

Each generator is provided with an enclosed ventilating system, complete with air ducts and surface coolers. Circulation of the cooling air through the ventilating system is accomplished by means of fans on the generator rotor. The warm air from the generator is discharged through eight openings in the generator stator frame, from whence it goes to eight surface-type air coolers and is then collected in a discharge housing and directed back into the machine from above the generator rotor.

The eight surface coolers are spaced symmetrically around the periphery of the generator stator. The cooling water is supplied to the coolers at a pressure of from 30 to 50 pounds per square inch

from the penstock. The coolers were specified to have sufficient capacity so that with one individual cooler out of service the temperature of the air entering the generator should be maintained at less than 40° centigrade, and the maximum indicated temperature of the generator maintained at less than 100° centigrade at rated generator output with the cooling water entering at 25° centigrade, using not more than 840 gallons per minute for each generator. The actual temperature of the cooling water is about 10° centigrade, giving a safe margin in the cooling system.

The shells for the surface coolers are fabricated of steel plates. The tubes, having an outside diameter of 1 inch, are made of Admiralty metal and the tube plates of Munz metal. Copper fins are spirally wound on and soldered to the tubes. The arrangement of the tubes is such that each tube can be removed individually and replaced. The water boxes are constructed with removable cover plates to permit access to all tubes for cleaning and inspection without disturbing the water pipe connections.

Surge protective equipment.

Surge protective equipment consists of lightning arrestors and capacitors connected to the main leads of each generator. The arrestors limit impulse voltages to approximately 37 kilovolts, which is considered safely below the impulse strength of the generator winding. The capacitors limit the rate of voltage rise above ground.

Generator neutral reactors.

A reactor is provided between each generator neutral and ground of suitable value calculated to reduce ground fault currents to the minimum necessary to assure positive differential relay action. The reactor is rated 0.82 ohm at 60 cycles, 8,000 amperes for one minute, 15,000 volts. It is of air-cooled construction and is enclosed in a steel housing. It is connected to the generator neutral through a single-pole oil circuit breaker rated 600 amperes continuous, 30,000 amperes interrupting.

Excitation and voltage control.

Each generator has a 275-kilowatt, 1,100-ampere, 250-volt, shunt-wound main exciter and a 15-kilowatt, 60-ampere, 250-volt compound-wound pilot exciter. Both exciters are direct connected and mounted at the top of the generator in a compact telescoped arrangement. The excitation system is designed to give a response ratio of 2 and a voltage ceiling of 550 volts.

The main generator voltage is controlled by varying the resistance in the main exciter field circuit, the resistance in the pilot exciter field having been set at a fixed value by means of a hand-operated rheostat. The main exciter field rheostat is motor operated and is normally actuated automatically by means of a voltage regulator, but under emergency conditions can be operated manually. Small voltage adjustments effected by the normal response contacts of the voltage regulator are made by small resistance steps in the motor-operated rheostat. Large adjustments in voltage are made by quick-response contacts of the regulator which energize high-speed contactors to cut in or shunt out large sections of the rheostat.

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The normal voltage level to be maintained by the regulator is controlled by means of the manual voltage adjusting rheostat on the main control board. Compensation for voltage drop through the step-up transformers may be obtained automatically by means of the voltage drop compensator which reduces the voltage applied to the voltage-sensitive element as a function of current and compensator setting. Reactive kilovolt-amperes are automatically divided between the two generators by means of an equalizing reactor, similar to the voltage drop compensator which operates to affect the voltage applied to the voltage-sensitive element so that the excitation applied to the generator carrying too much reactive is reduced and the other increased.

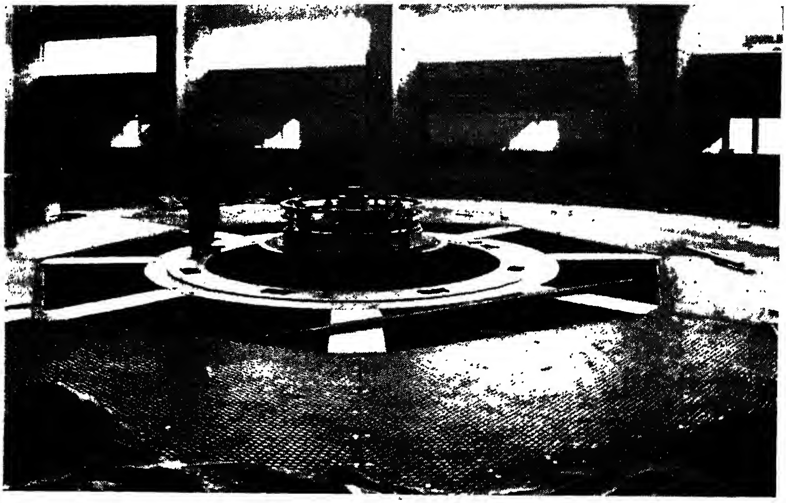

FIGURE 59.—*Exciters, main and pilot.*

Overvoltage is prevented by the automatic insertion in the pilot exciter field circuit of a fixed block of resistance. The resistance is manually shunted out when normal voltage is recovered.

Very low excitation, such as required for charging a long transmission line, is provided by a differential field winding on the main exciter which opposes the main field and is capable of effecting a slight negative exciter voltage.

Generator heaters.

To preclude the possibility of condensation within the generators during periods of shut-down, heaters are installed to maintain the temperature above the dew point. Each generator is equipped with eight 2-kilowatts, single-phase, 60-cycle, 440-volt heaters. The heaters are spaced symmetrically around the inside of the generator below the windings. The operating range of the heaters is from a minimum of 650° Fahrenheit to a maximum of 750° Fahrenheit.

Acceptance tests.

After complete erection, acceptance tests were made as follows:

ON EACH GENERATOR

1. Dielectric test of armature and field windings. Armature windings 28,600 volts for 1 minute. Field windings 5,000 volts for 1 minute.
2. Resistance of armature and field windings.
3. No-load saturation test.
4. Short-circuit saturation test.
5. Telephone interference factor determination.
6. Overspeed test.

ON ONE GENERATOR ONLY

7. Conventional efficiency test, including the determination of all losses.
8. Heat run to determine temperature rise under continuous load at rated output.
9. Deviation of wave form factor.
10. Test to determine direct axis transient reactance and negative and zero sequence reactances.
11. Test to determine short circuit ratio and synchronous reactance.

The conventional over-all efficiencies are as follows:

Load (percent rated kilovolt-amperes)	Efficiency, percent	
	Guaranteed	Actual
25	93.4	94.729
50	96.1	97.012
75	97.0	97.689
100	97.3	97.968

Generator switching and connections.

Each generator is switched and synchronized on the high voltage side of its step-up transformer bank. A motor-operated disconnecting switch is provided between the generator and the step-up transformer bank, and the station service transformer circuits are taken off the main leads between the disconnect and the step-up transformers so that station service may be taken from the low-voltage side of either step-up transformer when the generator is shut down if desired.

The generator leads consist of four 4-inch by ¼-inch copper bars per phase, mounted on porcelain supports and enclosed in sheet-metal housings with hinged access doors. These leads are carried to the electrical bay where similar housings are provided for instrument transformers, disconnecting switches, and surge protective equipment. From the electrical bay in the powerhouse to the transformer delta bus in the switchyard, a distance of some 400 feet, the leads consist of four 1,500,000-circular-mil, paper-insulated, lead-covered cables per phase, installed in 4-inch transite conduits embedded in concrete. The cables are arranged and transposed in such a way as to prac-

tically equalize the currents. The sheaths are grounded at about mid-point, and the ends, including the compound-filled terminals, are insulated to prevent sheath currents. The delta buses at the transformers are of copper tubing on outdoor insulators and steel structures.

MAIN TRANSFORMERS

A bank of three transformers connects each generator to the 154-kilovolt bus in the switchyard. Each bank is rated 56,000 kilovolt-amperes, 13.8/161 kilovolts, 55° centigrade, and is connected delta-wye. The neutral of each bank is grounded through a reactor. The transformers and the reactors are of the outdoor type, self cooled,

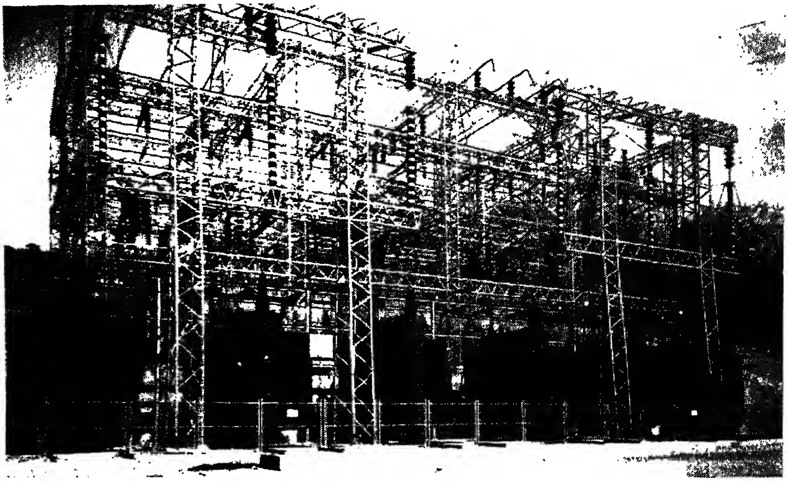

FIGURE 60.—*Main transformers and switchyard.*

oil insulated, with inert gas seal. The gas space above the oil is sufficient to permit expansion and contraction of the oil under usual loading cycles. A pressure alarm circuit indicates low gas pressure and a relief valve limits excessive pressure.

The transformers are designed to withstand a 5-second short circuit with full sustained primary voltage. A resistance temperature detector in the low-voltage winding of each transformer is connected to a Leeds and Northrup temperature recorder on the main switchboard which has contacts to announce high temperatures. Each transformer is also equipped with an oil temperature device with alarm contacts connected to the switchboard annunciator.

Each transformer is provided with one high-voltage, one neutral, and two low-voltage bushings. The high-voltage bushing is of the oil-filled porcelain type with glass oil-level indicator. The low-voltage and neutral bushings are solid porcelain type.

Each transformer has a supporting base equipped with wheels which rest on steel rails mounted on concrete piers in the switchyard.

The transformers are served by a transfer car and a motor-driven lift so arranged that the transformers can be rolled into the generator room under the crane on their own wheels, thus conserving headroom. Any indoor servicing or untanking of the transformers is accomplished by rolling the transformers to the generator main floor where they are handled by overhead cranes and placed in the erection floor space designed for electrical maintenance. The transformers were designed and manufactured by the General Electric Co.

Transformer neutral reactors.

The neutral reactor for each bank of transformers is of the outdoor type, single phase, self cooled, oil insulated, inert-gas sealed, and is mounted on a concrete pad adjacent to the power transformers.

Connections are made to the reactor through two exposed terminal bushings through the top of the tank. The neutral terminal is insulated for 34.5 kilovolts and the grounding terminal for 15 kilovolts. Copper tubes supported on insulators connect the reactor to the transformer bank neutral and the grounding terminal of the reactor is connected directly to the switchyard buried cable grounding system by bent bare copper tubes spaced away from the reactor tank.

A 25-kilovolt thyrite resistor mounted on the cover of each reactor connects the neutral terminal to the grounding terminal so as to limit any transient voltage stresses to values consistent with the insulation strength of the reactor windings. The steel reactor tank has a magnetic shield of short-circuited copper strap located between the reactor core and the tank wall to prevent excessive heating of the tank which might be caused by a magnetic field.

The reactor was designed by the transformer manufacturer specifically for use with this bank of transformers, and the transformers were impulse tested at the factory with the reactor connected in place.

SWITCHBOARDS AND CONTROLS

Centralized control for the power plant and switchyard is provided from a main switchboard located in a control room adjacent to the generator room. The switchboards include a bench control board and back-to-back vertical instrument and relay boards. Station battery distribution panels and station recorder panels are located, in the initial installation, adjacent to the instrument and relay panels, but space and building structures are so arranged that these panels may be later arranged in a separate board at the end of the control room if required by growth of the station. The control turret of the local and dispatching telephone systems is located on the operator's desk which commands a view of all boards except the relay board.

The generating units and auxiliaries are ordinarily started by the hydraulic operator and synchronized by the switchboard operator in the usual manner, but the control circuits are designed so that the switchboard operator can, if necessary, start the main units and auxiliaries with or without the aid of the hydraulic control operator. The generating units can likewise be stopped from the main control room, but the stopping of the unit auxiliaries must be done by the hydraulic operator.

The control room is air conditioned, has double glass along the generator room wall, and has an acoustical ceiling. Indirect lighting is provided by suspended fixtures.

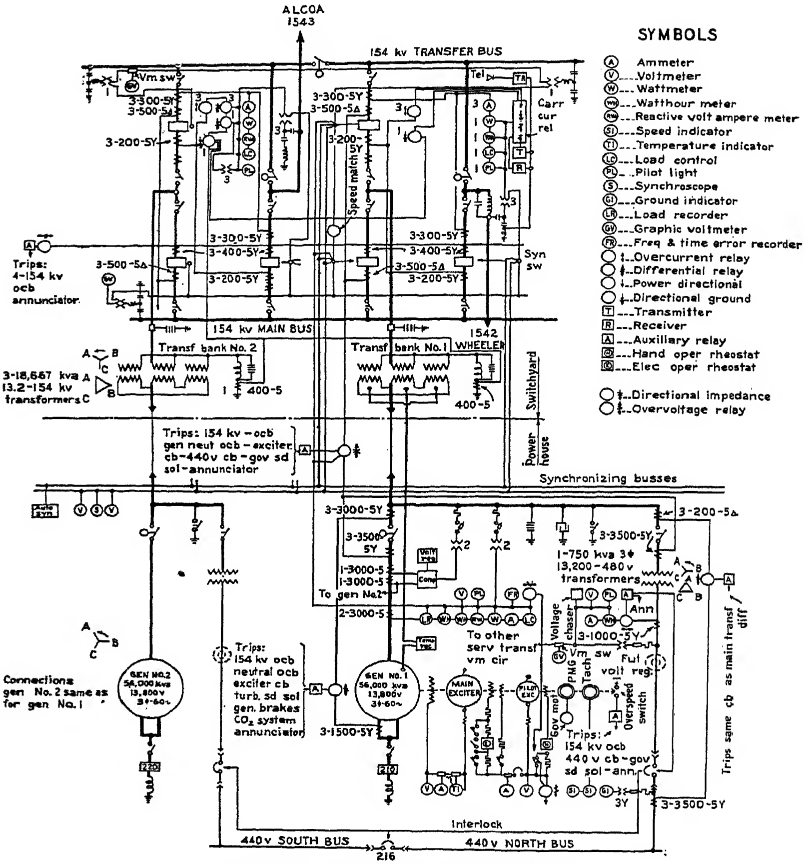

FIGURE 61.—*Single-line wiring diagram.*

Main switchboards and control wiring.

The benchboard contains controls for the two generators, the two main transformer banks, the high-voltage switchyard including the two 154-kilovolt feeders, and the two station service transformers. The main connections are indicated by miniature buses, using distinguishing colors for the various voltages.

The instrument switchboard contains instruments and meters for the above circuits and an annunciator above the main panels. Load and frequency control, water level recorder and indicator, generator

temperature recorders, and various recording voltmeters are also located on this board in the initial installation.

The relay switchboard contains protective relays for main station equipment, carrier current pilot relays for the 154-kilovolt lines, equipment for automatic synchronizing, and protective devices for miscellaneous circuits. Battery distribution switches are also located on this board in the initial installation.

All main switchboards are of dull-finished black steel. Wiring is enclosed in perforated steel grilles behind panel edges and carried downward to terminal boards enclosed in steel compartments located in a control terminal room immediately below the main switchboard.

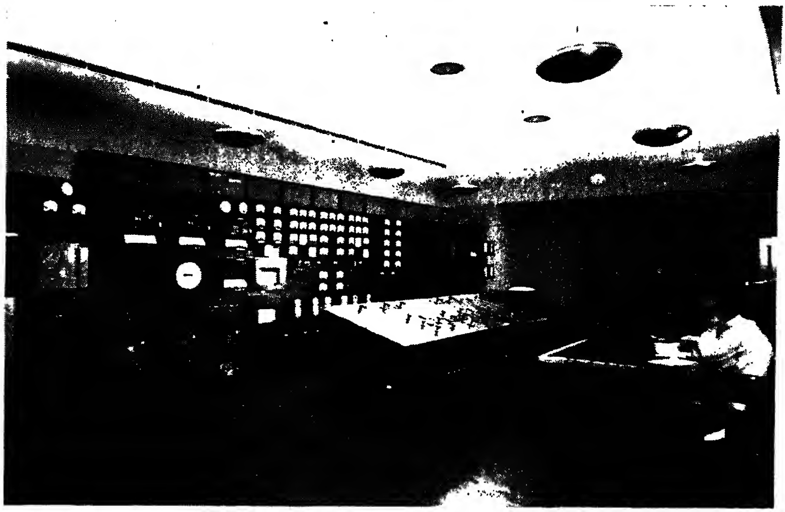

FIGURE 62.—*Main control room switchboards.*

Most of the inter-panel connections are made in the terminal board compartments, thus greatly simplifying the switchboard panel wiring. The terminal cabinets also afford a convenient connection between switchboard wiring and station cables.

The terminal compartments are arranged adjacent to cable trays which extend from the terminal room through the powerhouse and on through a control cable tunnel under the switchyard. From the trays the cables are carried through comparatively short conduits to their terminations in the powerhouse and the switchyard.

Unit exciter panels.

Steel panels are provided in the concrete foundation of each generator and contain main and pilot exciter rheostats, breakers, control relays, and necessary instruments for initial adjustments. They face the hydraulic operating aisle on the hydraulic operating floor.

Unit hydraulic gage boards.

A steel hydraulic gage board is provided for each generating unit facing the hydraulic operating aisle on the hydraulic operating floor. Each contains instruments for recording thrust and guide bearing temperatures and for indicating generator air and exciter air temperatures, thrust bearing oil level, and cooling water pressures and temperature.

Station service and unit auxiliary boards.

The station service and unit auxiliary boards are of the steel-enclosed, dead-front type. The station service board is located in the main control room and receives power at 460 volts from the two station service transformers and from a third emergency source. It controls feeder circuits directly to a number of the common station auxiliaries and to the unit auxiliary boards and other distribution centers. The unit auxiliary boards serve the generating units and are located on the hydraulic operating floor.

Station battery and control board.

The station battery is the sealed glass jar type, consisting of 120 cells mounted on wooden supports in the battery room. It has a capacity of 320 amperes for 8 hours or 40 amperes for 8 hours at 250 volts with final voltage of 1.75 volts per cell.

Two motor-generator sets are provided for trickle charging the battery and supplying the normal load on the battery bus. Each set has a 30-horsepower, 440-volt, 3-phase, 60-cycle induction motor directly connected to a 20-kilowatt diverter-pole type direct-current generator having a maximum voltage rating of 280 volts.

The control board consists of four steel panels of the dead-front enclosed type and is located in the east section of the powerhouse near the battery room. The board controls the station battery, the two battery charging motor generator sets, and the main distribution circuits.

Control of generators.

Indicating lamps are provided on the main control benchboard to show that the following conditions have been met before operating the gate control switch for admitting water to the turbine:

1. Cooling water is flowing through cooling coils of the oil reservoir for the thrust and guide bearing.
2. Cooling water is flowing through the generator air coolers.
3. Generator main exciter field circuit breaker is closed.
4. The governor oil pressure is within the normal operating range.

The starting control circuit for each generator is interlocked to prevent the starting of the unit unless the above conditions have been fulfilled. The gate control switch is operated in such manner that the turbine bearing oil pumps are started by the first position of the switch before the gates are opened by the second position.

The generator voltage regulator is normally left in service during the starting operation, and after the voltage builds up to normal the regulator automatically adjusts the generator voltage to the operating value for which the adjusting rheostat has been set. A transfer

switch is provided in the control circuit so that automatic voltage regulation may be changed over to manual if desired.

Automatic synchronizing equipment is provided. This equipment includes speed matchers with suitable protective features to prevent closing of control circuits unless the phase angle and voltage are correct. Under automatic synchronizing, the synchroscope and synchronizing voltmeters are energized so that the operator may observe the operation of the synchronizing equipment. Synchronizing may also be done manually.

Load and frequency control.

The load and frequency control includes all necessary equipment for the automatic control of the station to accomplish any one of the following conditions of operation within the limits of capacity of the generating units:

1. Maintain a selected tie-line load within predetermined adjustable kilowatt limits.
2. Absorb by the plant the load changes originating in its particular area so as to minimize unnecessary power transfer over the line.
3. Regulate the system frequency at 60 cycles with sufficient accuracy for satisfactory operation and timekeeping purposes.
4. Maintain a fixed output for station base load by means of unit control for regulating stream flow or other system conditions..
5. Hold on each generating unit a fixed load, adjustable at will, in order to permit the station to operate as two independent stations.

The control equipment is mounted on the main instrument switchboard and includes load control for the two main generators and for two 154-kilovolt lines, recorders for the two generators and the two 154-kilovolt lines, and one frequency and time error recorder.

Annunciator system.

The annunciator system provides audible and visible signals in the event of trouble or improper operation of certain designated devices. Certain of the signals are transmitted both to the main control room and to the hydraulic operating floor. Audible signals are automatically stopped at the end of approximately 10 seconds, but the visible signals, which are of the illuminated sign type, remain lighted until de-energized by the operation of a reset button or until the trouble contacts reset. A group light is provided at each panel section to guard against the loss of signals in the event of an individual lamp burning out. All indications can be periodically tested by the main control operator or hydraulic floor operator by operation of test buttons. Each annunciator is provided with means for sounding a bell alarm when a fuse in the individual annunciator circuit is blown.

Protective relays.

The principal relaying schemes are shown in the diagram of figure 61. The generator differential relays are balanced between a set of current transformers in the main leads and a second set of half capacity in half the neutral leads. They thus detect ground faults, open-coil faults, and coil-to-coil faults. The relays are designed for four-cycle operation on 0.2 ampere.

When the relays operate, the main oil circuit breaker connecting the unit to the high voltage bus is locked out and can be reclosed only after the differential auxiliary relays have been reset manually.

The differential relays for the main transformer banks operate from star-connected current transformers on the low-voltage side balanced against delta-connected current transformers on the high-voltage side. The relays are designed for four-cycle operation on 4 amperes. The function of the differential relays for the transformer is essentially the same as in the case of the generator in that auxiliary relays are energized to shut down the unit, but brakes are not applied to the generator rotor.

The differential relays for the two station service transformers energize auxiliary relays for trip circuits essentially the same as for the main power transformers. Separate auxiliary relays for the annunciator system show when the differential relays for these transformers operate.

The 154-kilovolt switchyard bus is protected by differential relays of the high-speed overcurrent type designed for four-cycle operation on 10 amperes. These relays operate from star-connected bushing-type current transformers in the 161-kilovolt oil circuit breakers. They energize auxiliary relays which are connected to trip all 161-kilovolt oil circuit breakers on the bus section. The breakers are automatically locked out until the differential and auxiliary relays are reset manually.

Carrier current relay system.

Carrier current relaying equipment is provided for protecting the 154-kilovolt transmission line from Norris to Wheeler, a distance of approximately 218 miles. The carrier equipment utilizes phase A and ground wire for the complete metallic circuit, with coupling capacitors connected to phase A on both ends of the transmission line. The relay group is so designed that the direction of power flow at one end of the line in relation to the direction of simultaneous flow at the other end of the line determines whether or not an electrical fault has developed on the protected line. The relay system will energize the oil circuit breaker trip circuits on both ends of the line in 0.05 second when a fault occurs but will not cause tripping of the breakers during instability of the system unless a fault occurs during the instability period.

The transmitter-receiver unit of the carrier relay system is mounted in a weatherproof housing located in the switchyard. Protective equipment includes a grounding switch, gap, and drain coil, which are connected to the line coupling capacitor.

The relay equipment is mounted on the relay switchboard in the main control room. Potential for the relays is supplied by resonant-type capacitor potential devices connected directly to the transmission line, and the corresponding current values are obtained from bushing-type current transformers on the 161-kilovolt oil circuit breakers.

The equipment is provided with a signaling arrangement for manually testing the carrier circuit. Equipment is also arranged to allow the use of a plug-in test telephone attachment for emergency communications between opposite ends of the transmission line.

Carrier current telephone system.

A carrier current telephone system is provided which uses phases B and C of the 154-kilovolt transmission line. The present carrier panel arrangement permits two-way telephone communication between Norris, Chickamauga, Wheeler, and Wilson. The system is used for general communication purposes, including load dispatching and for giving orders and instructions to transmission-line patrolmen who have patrol cars equipped with carrier-current telephone receivers. The selective ringing equipment is so designed that any extension or telephone station can be selectively called from any other station on the same carrier panel by the usual method of dialing.

Automatic telephone and signal system.

Local communication and signal facilities are provided by automatic telephone and signal equipment which is interconnected with the carrier-current telephone system and can also be connected to the public telephone system through a manually-operated, key-type switching turret. The key-type turret and exchange desk are located in the main control room in front of the benchboard, and the automatic switching equipment is located in the telephone room. The exchange provides for continuous day and night service without manual operation except for toll or other similar service.

Executive right-of-way service is provided on one line in the main control room which permits the head operator to make emergency or important calls to busy telephones by breaking in on established connections.

Conference service for a maximum of six telephones is provided for, the signal for conference calls being a steady, uninterrupted ringing of the called station. If the called station should be engaged an identifying tone signal is automatically put on the line to indicate that a conference is being called without interfering with the conversation.

A code call service of the usual type is provided. A call may be initiated from any telephone station by proper dialing for code call prior to dialing the station number. The code call equipment is mounted in the same cabinet with the automatic telephone apparatus.

Station clock system.

An accurately controlled 60-cycle frequency, 115-volt, 1,000-watt power supply is provided for driving all station clocks and recording instrument charts. The power supply equipment includes essentially a tuning fork, a motor-generator set, and control and distribution cubicle with accessory devices. The tuning fork is of the self-starting dynamic type driven by duplicate vacuum tubes. It is mounted in a constant-temperature case which makes the accuracy of the control devices independent of changes in room temperature. The design of the tuning fork is such that its accuracy is not affected by the voltage variation of the supply battery, and the output is such that when checked daily against Arlington time signals the error will not normally exceed 0.75 second (plus or minus) per day. The entire clock system is guaranteed to have an over-all accuracy such that the secondary clocks will have a variation from correct time not exceeding 2 seconds per day.

AUXILIARY POWER, LOW-VOLTAGE WIRING, GROUNDING, AND LIGHTING

Auxiliary power supply.

Power for station service and unit auxiliaries is obtained from two step-down transformers connected to the main generator leads between the disconnects and the step-up transformers. Each is rated 750 kilo-

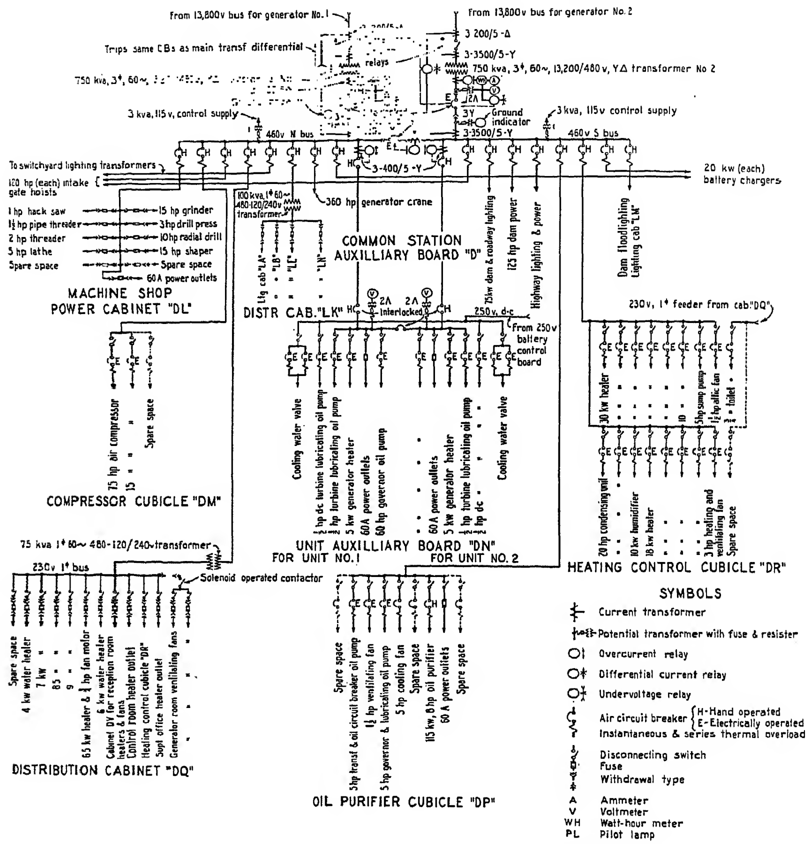

FIGURE 63.—*Station service single-line diagram.*

volt-amperes, 3 phase, 60 cycles, 13,200 volts (wye), 480 volts (delta) and are of the outdoor type, self cooled, oil insulated, and nitrogen gas sealed. They are located outdoors on the upstream side of the powerhouse in individual recesses of the powerhouse building structure.

To provide against concentrating the capacity of both auxiliary transformers on the station service bus at one time, which would require high interrupting capacity in the breakers and also might

result in objectionable cross currents between generators, the two transformer breakers at the 460-volt bus are interlocked with the 460-volt bus tie breaker so that not more than two of these three breakers can be closed at once. This interlock is also carried on throughout the auxiliary power circuits as is necessary to prevent paralleling the two auxiliary power transformers anywhere in the station. The interlocks are all arranged so that in case transfer of load from one bank to the other is necessary, the load must be disconnected instantaneously, as this is believed to involve less operating hazard than would result from paralleling momentarily.

The auxiliary switching is from switchboards of the dead-front or cubical type, equipped with carbon circuit breakers. The station auxiliary board provides nine circuits for the principal powerhouse and dam operating equipment as follows:

1. Unit auxiliary board.
2. Crane in generator room.
3. Dam power.
4. Intake gate hoists.
5. Battery charging sets.
6. Compressors.
7. Electrical heating and air-conditioning.
8. Station lighting.
9. Miscellaneous distribution and lighting.

Since the two station-service transformers are connected to the main leads on the transformer side of the generator disconnecting switches, they may be energized when the generators are shut down and disconnected. The usual operation is with the 460-volt bus tie breaker closed and the entire load supplied from one or the other but not both of the station-service transformers. To provide auxiliary power, lighting, and heating when neither generator is needed for generation or for condenser operation and when it is not desirable to energize a main transformer bank for the small station-service load, a small station-service emergency transformer is provided to take energy from the tertiary of the autotransformers in the switchyard.

Low voltage wiring.

Cables for auxiliaries, control, and miscellaneous indoor station equipment are insulated for 1,000 volts with Performite rubber compound, having at least 35 percent by weight of best-grade new rubber, the compound being highly moisture resisting, superaging, and capable of operating continuously with a maximum copper temperature of 75° C. The cables are braid covered with moisture-resisting, flameproof, cotton braid except where the cables are subject to excessive moisture, such as for the dam and for the switchyard, in which cases the cables are lead covered. Vertical risers in the dam are steel armored. Cables to the switchyard are run on cable trays through a concrete tunnel and thence in galvanized steel conduits to the equipment. Power and control cables for the dam are run in galvanized steel conduits supported on racks in the inspection and operating galleries, but where the conduits pass from the powerhouse to the dam they are embedded in concrete. Wherever practi-

cal to do so, all steel conduits for power control and miscellaneous cables within the generating station are embedded in concrete walls, floors, and foundations.

Grounding system.

The grounding system for the power plant, dam, and switchyard consists of a network of bare cables connected to two main grounding mats. One ground mat is located on the upstream side of the dam

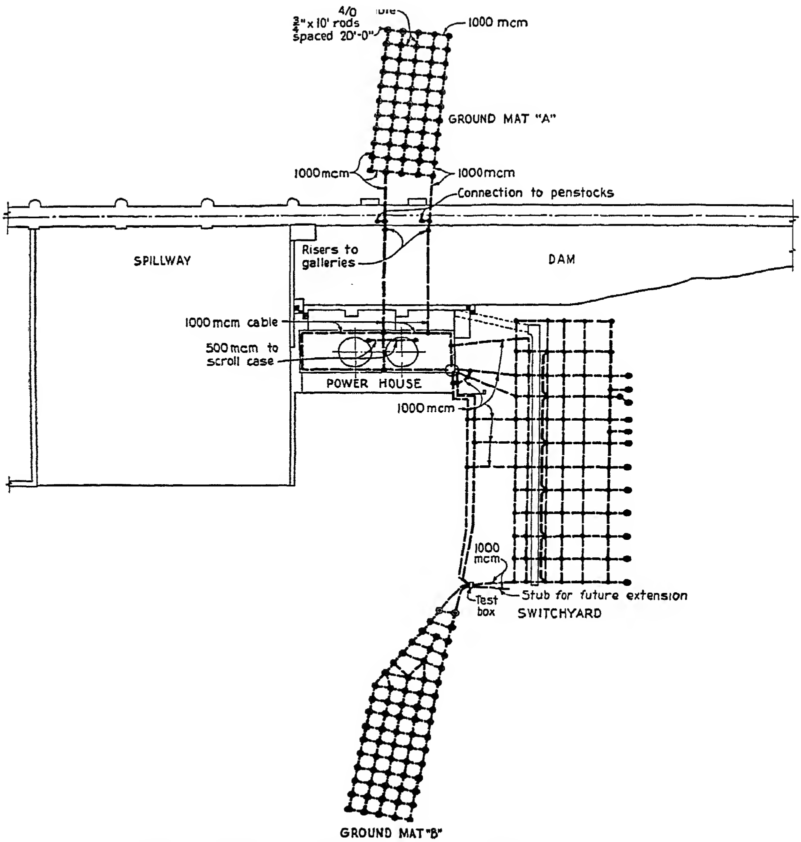

FIGURE 64.—*Grounding system for dam, powerhouse, and switchyard.*

and connected to the steel penstocks and scroll cases, and the other is in filled ground under the parking area downstream from the powerhouse. The mats are of bare copper cables thoroughly brazed at all cross-connections and to numerous copper-weld rods. The brazed connections were thoroughly painted with asphaltum before being

covered or buried. The cables connecting the grounding system to the mats are rubber insulated and provided with test stations at which periodic ground resistance tests can be made.

Lighting.

The lighting system is 115/230-volt, single-phase, supplied from a 100-kilovolt-ampere transformer and distributed through four lighting cabinets. An independent 250-volt direct-current emergency system with automatic control is available at all important operating points and in passages and stairways.

The generator room lighting is provided by twenty 750-watt high-day, direct-type commercial lighting units. The main control room is lighted by eight 1,000-watt totally indirect units. The reception room is provided with fixtures of special design harmonizing with the architectural treatment of the room. It includes a rectangular, totally indirect unit in the center for general illumination and louvered strip lights with control lenses for direct illumination of murals and wall displays. The remaining powerhouse lighting fixtures are of the standard types usually employed for similar service, with a few cases of semi-special units adapted to local conditions.

The exterior of the powerhouse is provided with special wall fixtures with control lenses for local illumination. The soffit over the main entrance is provided with recessed flush-type ceiling fixtures with stippled diffusing lenses.

The roadway on top of the dam is lighted by low-type lighting units of a special design which serve also as supports for the handrails. The units, which are mounted on approximately 14-foot centers, are provided with 100–watt lamps, control lenses, and louvers so arranged optically as to direct the light output toward the roadway with practically complete cut-off above the horizontal. A dense opal glass panel on the back of each unit provides outline lighting which is clearly visible both upstream and downstream from the dam. Control is by photoelectric relay.

The galleries in the dam are lighted by fixtures supported by exposed conduits clamped to the gallery ceiling. Fifty-watt lamps, spaced approximately 20 feet apart, are installed in an upward position in cone-shaped recesses in the concrete which are painted white to serve as effective reflectors.

The switchyard is lighted with 300-watt Holophane substation fixtures mounted on brackets from the steel structures. A small group of the lights is automatically controlled by a photoelectric relay.

Twelve 1,000-watt floodlight units illuminate the downstream face of the dam, spillway, and powerhouse.

POWER PLANT AUXILIARIES

Water level gages.

Four water-level gages of the selsyn type are provided for continuously indicating and recording headwater and tailwater levels. Three gages are installed for indicating and recording the headwater elevations, two in float wells which are connected to the reservoir inside of the two trashrack structures, and one in a well connected to

the reservoir outside of and between the two trashrack structures. The fourth gage is installed in a well which is connected to the tail water in the tailrace.

Each gage consists of a float, tape, and balanced counterweight mechanism for transmitting the vertical motion of the float; a selsyn motor transmitter; and a counter-type indicating device. The recorders and remote indicators consist of a selsyn motor receiver, a counter-type indicator, and an electric clock driven strip chart.

The stilling wells for each of the gages are formed in the concrete of the dam for the three headwater gages and in the powerhouse substructure for the tailrace gage.

The receivers and recorders are mounted on the instrument board in the main control room.

Oil purifier, storage, and piping system.

Only two types of oil are used, transil oil for the transformers and oil circuit breakers, and governor and lubricating oil. A separate distributing system is provided for handling each type to and from the equipment.

The oil purifier is of the combination centrifuge and filter press type, manufactured by DeLaval Separator Co. It is a self-contained unit mounted on a common base plate with centrifuge, centrifuge motor, dirty-oil pump, clean-oil pump, strainer, electric heater, automatic temperature control, thermometers, filter press, electrically controlled gages, sampling cock valves, fittings, and piping.

The centrifuge is capable of purifying transformer and circuit-breaker oil containing 1,000 volumes of water in 1,000,000 volumes of oil in a single pass at a temperature of not over 50° C., and delivering 1,200 gallons of purified nonaerated oil per hour containing less than 5 volumes of water in 1,000,000 volumes without the aid of the filter press. The centrifuge is also capable of purifying the same quantity of lubricating and governor oil at a temperature of not over 60° C. The filter press is capable of removing fine particles of carbon or dirt from transformer or oil-circuit-breaker oil at the rate of 1,200 gallons per hour. The heater is capable of raising 1,200 gallons per hour of transformer and oil-circuit-breaker oil from minus 12° C. to plus 50° C. or 600 gallons per hour of lubricating oil from minus 12° C. to plus 60° C. in one pass.

A portable dielectric testing set and a filter paper drying oven are also included in the oil purifier equipment.

Two oil pumps serve to transfer the oil for the oil circuit breakers, transformers, governors, and lubrication. The pump which handles the transformer and circuit-breaker oil is a herringbone-gear-type rotary pump, with a capacity of 200 gallons per minute at 70 pounds per square inch pressure, and is directly connected by a continuous spring flexible coupling to a 15-horsepower motor. The pump for the governor and lubricating oil is a gear-type rotary oil-pumping unit with a capacity of 38.6 gallons per minute at 20 pounds per square inch pressure and is direct connected to a three-horsepower motor. Each pumping unit is equipped with an unloading device.

Five tanks are provided for oil storage as follows:

Tank No.	Function	Capacity
		Gallons
1	Unfiltered transformer oil	12, 000
2	Unfiltered circuit-breaker oil	8, 000
3	Filtered transil oil	12, 000
4	Filtered governor and lubricating oil	2, 400
5	Unfiltered governor and lubricating oil	2, 400

All of the tanks are of all-welded steel construction and rest on reinforced concrete saddles.

The oil-purifier equipment, oil-testing equipment, and pumps are located at the east end of the powerhouse at elevation 847. The storage tanks are in an adjacent room on the same floor level.

Compressed air system.

Two stationary air compressors with after-coolers and air receivers are included in the plant equipment. These furnish compressed air at 100 to 125 pounds per square inch gage pressure for operating generator brakes, actuator, miscellaneous pneumatic tools, and furnishing compressed air for trashracks and cleaning lines. Both compressors are of the stationary, single-stage, horizontal straight-line, center-crane type. The larger of the two stationary compressors has a capacity of 330 cubic feet of free air per minute and is driven by a 75-horsepower motor. The smaller of the two stationary compressors has a capacity of 25 cubic feet of free air per minute and is driven by a 15-horsepower motor. A small portable compressor for furnishing air at a pressure of 300 pounds per square inch to the governor system is also included in the plant equipment. This compressor has a capacity of 8 cubic feet per minute and is driven by a 3-horsepower motor. The two stationary compressors are located on the elevation 847 floor, adjacent to the machine shop area.

Traveling crane.

A Harnischfeger crane of the indoor overhead traveling type is provided in the main powerhouse building for handling the heavy parts of the generator and turbine during erection, maintenance, or repairs. It is electrically driven and equipped with two trolley carriages, each provided with a main and an auxiliary hook. The rails on which the crane operates are located on top of the main powerhouse structural steel columns at elevation 893. Power is supplied through copper angle conductors located on the side of the crane girder below the runway rails. The rated full-load capacity of the crane is 250 tons, exclusive of the lifting beam, which is utilized in handling the large pieces of equipment. Each auxiliary hoist has a capacity of 20 tons. The travels and speeds are as follows:

	Travel in feet	Speed feet per minute
Main hook		5
Auxiliary hook		35
Trolley cross travel		30
Bridge travel	[1]	105

[1] Length of powerhouse.

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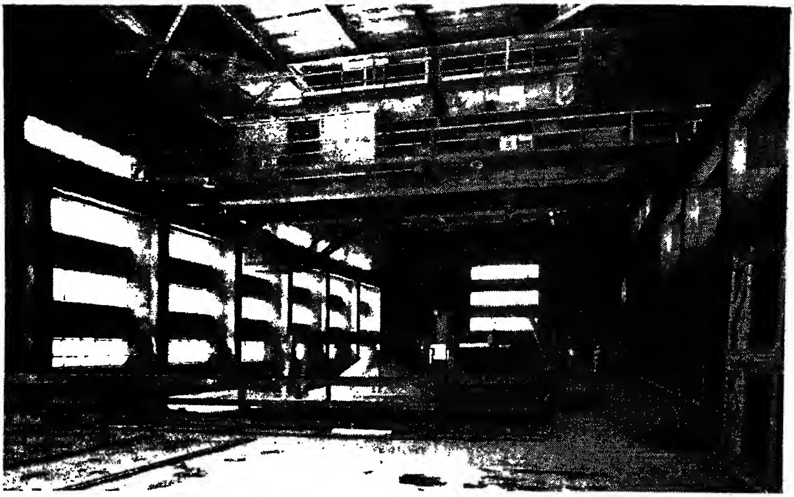

FIGURE 65.—*Powerhouse crane.*

Air Conditioning

An all-year air-conditioning plant is provided for the visitors' reception room and toilets, the administration office, and the control room. The cooling unit consists of a central cooling coil through which reservoir water at a temperature between 40° and 50° F. is circulated under control of a solenoid-operated valve. The heating unit consists of a central blast heater and a humidifier. The air-distribution system is through galvanized sheet steel ducts and circulation is accomplished by a main distribution fan, two exhaust fans, and dampers and grilles. The entire system is automatically controlled.

The reception room, office, and control room are served by two separate systems of sheet-iron ducts. One carries conditioned air from and the other returns air to the motor-driven fan from the main air-distribution plant. The toilet rooms are indirectly conditioned, air from the reception room being drawn through grilles in the connecting doors and then exhausted to the atmosphere through ducts by a motor-driven fan. The air returning to the main air-conditioning plant is mixed with fresh air in relative proportions, as regulated by manually operated mixing dampers at a point just ahead of the cooling unit on the suction side of the fan. The fresh air is drawn through a duct from a grille located in the parapet wall on the north side of the building.

The central blast heater consists of a number of finned electric strip heaters set into a rigid steel frame and installed in the sheet-metal duct. The capacity of the heater is 72 kilowatts divided equally between four circuits of 18 kilowatts, each delta-connected for 440-volt, 3-phase service. The humidity of the conditioned air is regulated by discharge into it of live steam generated in an

electric heater. The humidifier is under the automatic control of a humidistat.

Air for the generator room is cooled by a surface cooler through which water drawn from the reservoir at a temperature of between 40 and 50° Fahrenheit is circulated. The circulator fan draws air across the surface cooler and discharges the cool air into the pipe trench in the elevation 847 floor and thence through the building and back to the fan room through two large grilles in the west wall of the fan room. Four exhaust fans located near the ceiling of the powerhouse building in the upstream wall handle the discharge of air from the building. The fans are automatically started by a differential switch when the inside air becomes 5° Fahrenheit warmer than the outside air provided the inside air is above 60° Fahrenheit.

Exhaust from the oil-purifier room, oil-storage room, battery room, and the room in which the carrier current, frequency, and battery-charging sets are located is handled by another exhaust fan mounted in the oil-purifier room. This fan is manually controlled and operates continuously. The generator room circulating fan and the exhaust fan in the oil-purifier room are automatically shut down whenever the carbon dioxide system operates for either the oil-storage or purifier room. Fire louvers are provided in the two oil-storage room intakes, in the west wall of the oil-storage room, in the oil-purifier room intake, and in the wall between the oil-storage and purifier rooms. Another fire louver is provided on the oil-purifier room discharge. The fire louvers are automatically closed upon operation of the carbon-dioxide system for the oil-storage or oil-purifier room.

A separate ventilating fan and system of ducts and grilles handles the ventilation of the employees' locker room and toilet, telephone room, station-service room, and oil circuit breaker No. 2 room. The exhaust for this system is in the west wall of the janitor's room. The ducts for this system are carried in the space between the ceiling and the roof.

Water supply.

The raw water supply is obtained from the two penstocks by a 14-inch pipe attached to each penstock at elevation 832. The water passes through a duplex strainer and thence to two main pipes, one of which enters the main pipe trench where a cross is provided from which three other lines lead off. The other main pipe supplies water to the eductors. The three lines branching off from the cross in the pipe trench serve to convey cooling water to the two generator cooling systems, the turbine guide bearing, the thrust bearing and upper guide bearing, the air conditioning, the cooling and ventilating system, and also supplies water to the turbine seal rings when the unit is motoring. A surge line takes off the line to the trench and extends to the roof of the powerhouse.

The domestic water supply for the power plant is obtained from the town of Norris water supply reservoir through a 3- and 4-inch service main. The service line enters the powerhouse through the east wall of the oil-purifier room near the ceiling and tees into a line which extends on one branch of the tee to the overhead fire protection sprinkler system in the oil storage room and on the other

branch to a valve pit in the floor of the oil purifier room. In the valve pit, the four-inch high-pressure line connects to a pressure-reducing valve which reduces the pressure to 60 pounds per square inch for distribution as drinking water, as a cooling water supply for the station compressed air plant, and to the sanitary facilities. A bypass is provided around the pressure-reducing valve for emergency purposes in case of the failure of the pressure-reducing valve. The 4-inch high-pressure line also continues from the valve pit in the main pipe trench from which pipes lead to the fire cabinet and the overhead sprinkler system in the fan room and oil purifier room. The 4-inch high-pressure line is reduced to a 3-inch line after it leaves the pipe trench near the west end of the powerhouse and extends to a 20,000-gallon capacity fresh water tank on top of the hill on the west bank of the river. The pipe extends to the 880 gallery of the dam through the 880 adit and thence along the gallery wall to the top of the dam. It runs from there in a trench to the tank. The tank serves as an emergency water supply in case of failure of the main supply line from the Norris town reservoir.

Sewage disposal.

Disposal of the sewage from all the sanitary facilities is by a single septic tank located adjacent to the downstream east corner of the powerhouse. The tank discharges into the tailrace through a 6-inch soil pipe.

Fire protection equipment.

A carbon dioxide fire extinguisher unit is provided in the powerhouse for protection against fire in the two generators, the oil purifier room, and the oil storage room. The system is composed of two independent banks of carbon dioxide cylinders.

The system is arranged for either thermoelectric, push-button electric, or manual release to the 2 generators, each of which has an air space of 10,700 cubic feet. The equipment for each 20-cylinder bank is arranged for an initial discharge of 10 cylinders and a series of 5 delayed discharges of 2 cylinders each to either generator. The equipment is also arranged so that by means of a throw-over switch either bank can be placed in readiness for automatic operation for the protection of the generators and the other bank placed in reserve. Additional equipment is provided for release by push-button electric or manual control of 10 cylinders from 1 bank to the oil purifier room with an air space of 12,500 cubic feet and 10 cylinders of the other bank to the oil storage room which has an air space of 15,500 cubic feet. The throw-over switch for the automatic operation to the generators does not affect this equipment.

In addition to the carbon dioxide system, an automatic overhead sprinkler system is provided in the oil purifier, oil storage, and fan rooms. The water supply for the sprinkler system comes from the high-pressure domestic water line. The sprinkler system supplements the carbon dioxide system.

Fire louvers and fire doors are provided to supplement the fire extinguisher equipment. Fire louvers are provided in the ventilating intakes of the oil storage rooms and in the ventilating intakes

of the oil purifier room. Fire doors are provided for the two doors to the oil storage room.

Portable equipment.—The switchyard is provided with two portable fire extinguishers, one on each oil circuit breaker level. The extinguishers are 40-gallon capacity, truck type, foamite, manufactured by the American La France & Foamite Industries, Inc. They are housed in small concrete structures heated by 1-kilowatt electric heaters to prevent the foamite from freezing in cold weather. The trucks are equipped with large diameter wheels for easy transportation to any part of the switchyard.

Machine shop.

The machine shop is located in the upstream west corner of the powerhouse, adjacent to the erection bay and is not enclosed. The equipment includes a lathe, a pipe threader, an upright drill, a sensitive drill, a grinder, a hack saw, and a shaper. It is sufficient to handle all necessary machine shop work for normal maintenance of the plant.

POWERHOUSE SUBSTRUCTURE AND SUPERSTRUCTURE

The substructure of the powerhouse is composed largely of mass concrete founded on solid rock. The walls included in the substructure are all of reinforced concrete. The draft tubes for the two turbines are formed in the concrete of the substructure. Reinforced concrete piers support the draft tube roof slab and the walls and deck above the draft tube roof which form the stop-log deck and storage compartment. The upstream end of each pier is formed by a pier-nose casting of grade 2 (medium) cast iron.[18] The substructure concrete also surrounds the scroll case and the connections to the penstocks.

Access to the draft tubes is through galleries formed in the concrete of the substructure upstream from the draft tubes. The anchor bolts for the structural steel of the powerhouse superstructure are embedded in the substructure concrete.

The design of the concrete portion of the powerhouse was based on the concrete having an ultimate compressive strength of at least 3,000 pounds per square inch and the sum of the combined working stresses not exceeding:

	Working stresses in pounds per square inch
Tension in reinforcing steel	18,000
Compression on the extreme fiber of concrete	1,050
Same, but adjacent to supports of continuous or fixed beams or of rigid frames	1,200
Punching shear on concrete	225
Shear or diagonal tension	60
Bond on deformed bars	150
Bond on plain bars	120
Bearing on concrete:	
Load over entire area	750
Load over partial area—maximum	1,050
Bearing on rock	40,000

The construction joints in horizontal planes were designed so that the area of the keys would be sufficient to carry the total horizontal

[18] Federal Specification QQ-S-681.

shear without exceeding a stress due to punching shear on the keys as specified above but in no case were the keys to constitute less than 20 percent of the total area of the horizontal surface. Keys were proportioned on the assumption that frictional resistance at a coefficient of 0.5 was available on planes where the normal load was compressive.

Sufficient horizontal reinforcement was placed in the draft tube piers to sustain the bending stresses induced by the resultant of a moment caused by the horizontal shearing force acting at the elevation of bedrock. Reinforcement in the draft tube was proportioned for a gross hydrostatic uplift of 10 feet.

The superstructure is composed of a framework of structural steel, encased in reinforced concrete, and reinforced concrete walls. The floors are reinforced concrete slabs resting on structural steel concrete-encased floor beams. The roof is carried on steel Howe trusses supported by column extensions attached to the crane columns designed to support the crane girders as well as the roof. Equipment loadings used in the design of the floors and substructure were:

	Load in pounds
Weight of generator (complete)	1,672,000
Weight of generator rotor	500,000
Weight of generator shaft	52,000
Weight of water wheel (complete)	1,225,000
Weight of water wheel runner and shaft	165,000
Thrust load on water wheel	500,000
Generator short circuit torque	1,360,000
Weight of transformer	160,000
250-ton crane	[1]

[1] 8-wheel concentrations of 80,000 pounds each, on each rail, spaced 3 feet, 4 feet 3 inches, 3 feet, 6 feet 3 inches, 3 feet, 4 feet 3 inches, 3 feet. No impact allowance was included in these concentrations in the designs of the substructure as impact will be dissipated before reaching the substructure.

Floor live loadings used in design were:

	Load in pounds per square foot
Floor at elevation 830 under assembly bay floor	300
Assembly bay floor at elevation 847	1,000
Unloading bay floor at elevation 866	1,000
Machine shop floor at elevation 847	300
Electrical bay floors at elevations 855 and 866 (also reception room floor)	150

The crane runway girders were designed for a 250-ton crane, plus 10-percent impact, plus side thrust at each wheel of 6,500 pounds, plus a longitudinal thrust of 32,000 on the entire girder.

The columns for the powerhouse superstructure were designed for a live and dead crane load of 425,000 pounds, live and dead roof load of 131,000 pounds, wind load of 500 pounds per foot of column, and a crane side thrust of 31,000 pounds, or a combination live and dead roof load, plus 33 feet of water load, plus a wind load of 200 pounds per linear foot. In this latter combination the stresses were allowed to go just below the elastic limit of the steel. Crane live load was not considered.

The roof was designed for a live load of 75 pounds per square foot, plus a dead load of 95 pounds per square foot.

On the outside of the concrete walls, an appearance of matched-joint, large, stacked stone has been obtained by using lumber rough-sawed on one face for lagging with the direction of the lagging and alternated from horizontal to vertical in adjacent blocks. The block

appearance is emphasized by what appear to be joints between blocks, obtained by using V-shaped strips to separate adjacent blocks. The inside of the concrete walls is finished smooth. The outside of the walls received no treatment other than that produced by the board marks which are lightly visible and give a very pleasing appearance.

The window arrangement is somewhat unusual to customary practice for box-shaped powerhouse structures. The windows are arranged with their longest dimension in a horizontal direction rather than vertical. In the downstream wall, the windows are arranged in four horizontal rows, with eight windows in each row.

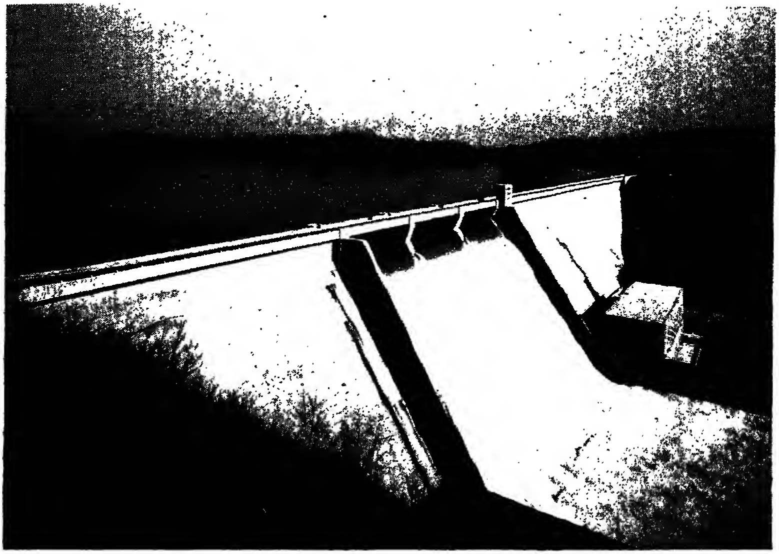

FIGURE 66.—*Architectural treatment.*

A wall space about equal in vertical dimension to the vertical dimension of the window opening separates each two horizontal rows of windows. The same general window arrangement is followed on the west wall and the upstream wall of the powerhouse. The bottom row of windows on the downstream wall is equipped with six projecting aluminum sashes with hand-operated mechanical window openers. The glazing for all windows is ⅛-inch-thick grade A Coolite ribbed glass, except the bottom section at the east end of the downstream wall, which has polished clear glass.

The opening in the east wall of the building is provided with a removable transom and double swinging doors built entirely of aluminum and glazed with polished clear glass. The opening, which is 18 feet 2 inches wide and 31 feet high, provides clearance for the movement of the largest piece of equipment that will be moved into or out of the building.

Small doors in the powerhouse are of all-aluminum frame with either clear glass, opaque glass, or fixed louver-filled openings depending on the location and function of the door.

The operating floor and machine shop floor at elevation 847 and the floor of the balcony and unloading bay at elevation 866 is brick-red quarry tile with quarry-tile base. The floors of the visitors' reception room and toilet and the employees' locker rooms are of terrazzo with precast terrazzo base. The main control room, the telephone equipment room, and the administration office floors are covered with linoleum on concrete. The other floors in the powerhouse are "steel trowel finish" concrete.

FIGURE 67.—*Visitors' reception room.*

The exposed portion of the structural steel framework of the powerhouse building is finished with a dull-gray paint. The steel was given the usual red-lead shop coat at the fabricator's plant, and after erection a patch coat of red lead was applied to all field-driven rivets and surfaces where the shop coat had been damaged in handling. A primer coat was applied to the entire exposed surface prior to the application of the finish of dull gray. In the remainder of the powerhouse building, except where plain concrete surfaces were left untreated, the walls and ceilings were given special finishes to suit the requirements.

The powerhouse, control room, and reception room have flat roofs built of precast concrete slabs laid on purlins, over which is placed a five-ply built-up Johns Manville 20-year-guarantee roof. Above the built-up roof, another layer of precast concrete slabs is arranged so that they rest on precast concrete discs, 10 inches in diameter and two inches thick, which in turn rest on the built-up roofing. The air space between the top slab and the built-up roofing is desirable in preventing

condensation and as heat insulation. The roof of the powerhouse is exposed to view from the roadway across the top of the dam and from the surrounding hills and overlooks. It was desirable to finish it in a manner that would harmonize with the powerhouse walls and the dam, and for this reason the concrete slabs were used.

The early powerhouse designs included two outlet conduits, each provided with a separate inlet structure, an emergency gate, and an 84-inch needle valve. These conduits were located in block 36 just west of the turbines in the powerhouse. The conduits would have served the purpose of releasing small flows through the dam, and in conjunction with the main outlet conduits, in securing close regulation. Because close regulation of flow could be obtained either with the slide gates or through the turbines, these conduits, together with their needle valves, slide gates, and intake structures, were deleted from the final design, and the affected portion of the powerhouse rearranged.

Through omission of the butterfly valves in the penstocks the powerhouse was moved upstream 22 feet 6 inches. Further studies indicated that the superstructure could be reduced in height and width. The height was reduced 27 feet 11 inches.

The generator room floor in the powerhouse, together with the protective powerhouse wall, was lowered 14 feet to elevation 866. They had originally been designed for a tail water level of 880. Further study of gaging records below the dam indicated that occurrence of levels above elevation 866 would be too infrequent to require consideration and that the normal tail water should be established at elevation 826 instead of 833 as originally assumed. Accordingly the draft tube deck was lowered from elevation 847 to 836.

SWITCHYARD

The switchyard is located on the hillside east of the powerhouse and downstream from the dam. It covers an area 132 feet wide and 180 feet long, and is so arranged that future extensions can be made downstream from the present installation. The hillside location was conveniently adapted to a three-level arrangement. The main transformers and neutral grounding reactors are located on the lowest of the three levels for convenience in handling. The oil circuit breakers are located on the other two levels.

The transformers, oil circuit breakers, and neutral reactors rest on reinforced concrete pads, and the structural steel is supported on concrete footings. The surface of the yard is of crushed stone. The tunnels which house the cables, connecting the generators with the transformer banks, and also the control cables, are of reinforced concrete construction. The transformer-transfer handling equipment is all outside of the switchyard area proper and all rails are flush with the roadway. The transfer car lift pit, also flush with the roadway, is on the longitudinal center line of the powerhouse.

Structures.

The principal concrete structures in the switchyard are the cable tunnel, the structure footings, and the equipment pads. Miscellaneous concrete structures include the stairways, walkways, and minor retaining walls.

The cable tunnel is composed of two parts, a single tunnel connecting the switchyard with the powerhouse and a double-deck tunnel which extends the entire length of the yard parallel to the longitudinal center line of the yard. In the design of the pull room walls and bottom slabs, investigations were made using earth and

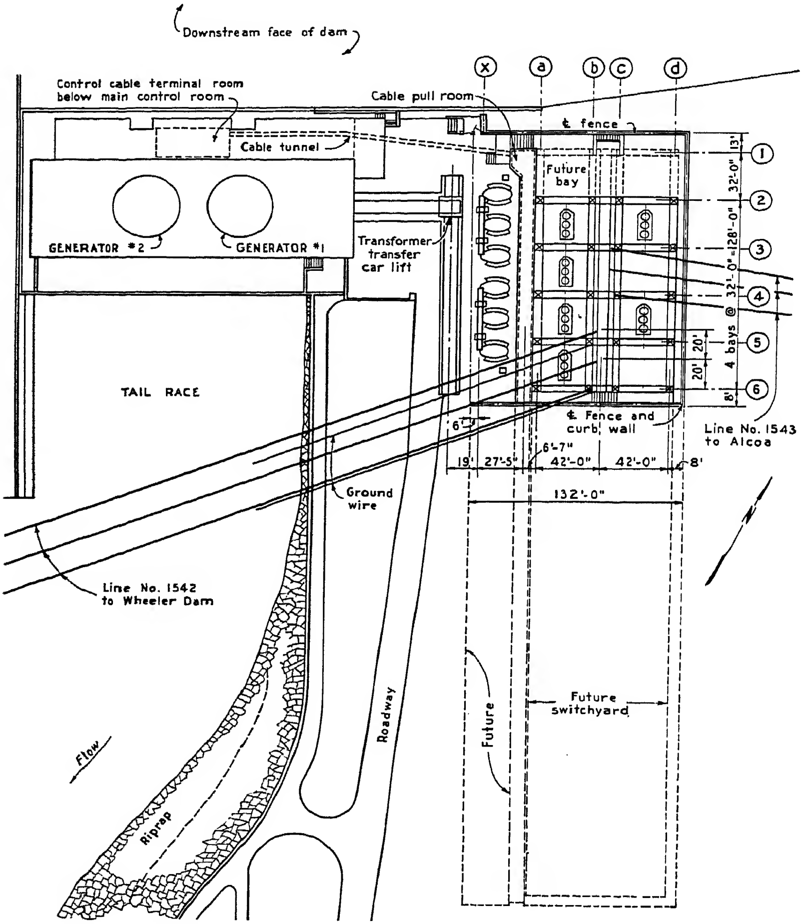

FIGURE 68.—*Switchyard—Location plan.*

water loads on the hillside together with uplift on the base. The structure was designed as a rigid frame. Since the tunnel acts as a retaining wall for the area east of the tunnel, anchorage against overturning was accomplished by means of vertical anchors into the rock.

All footings for the structural steel are reinforced by means of vertical and hook reinforcing bars. In order to resist the combined column thrust and uplift, all footings are anchored by means of bars, wedged and grouted in drill holes to a minimum depth of 10 feet into solid rock. The bottom of all footings is carried to solid rock. In the cases where footings exceed 8 feet 6 inches in height, larger footings 5 feet 2 inches by 6 feet 6 inches are provided below the maximum limit.

The transformer-transfer car track, which extends from the powerhouse to the lift pit, has a solid concrete substructure extending to rock.

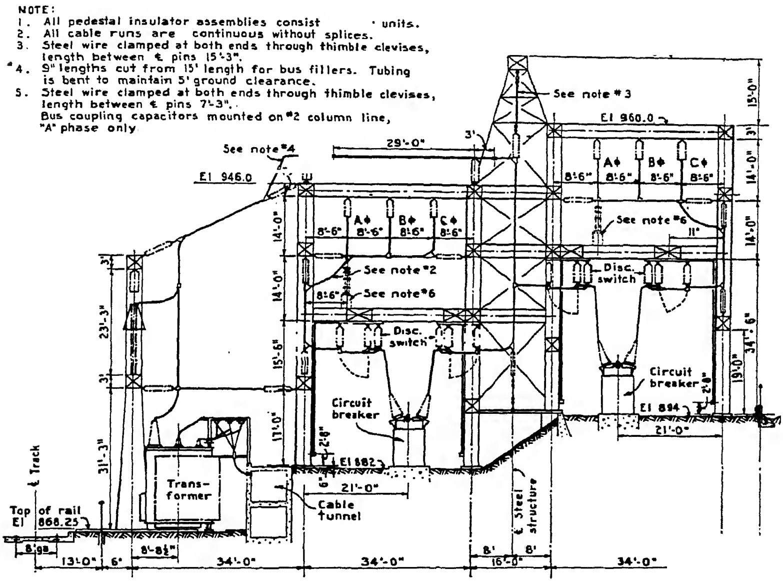

FIGURE 69.—*Section—154-kilovolt switchyard.*

The bus and switch supporting structure is the conventional latticed galvanized steel column and girder construction, with switchyard towers in the form of A-frame extensions attached to the top of the main structure. The connections, both field and shop, are all bolted.

Equipment.

The oil circuit breakers are each rated 600 amperes, 161 kilovolts, 60 cycles, with an interrupting capacity of 1,500,000 kilovolt-amperes. The breakers are single throw, 250-volt direct-current control, solenoid operated, trip free, and fully automatic. Each phase is in an individual tank, with all poles of one breaker unit-operated by a common enclosed mechanism. The three tanks of one unit and the solenoid mechanism are mounted on a common concrete pad. The breaker

tanks and covers are made of boiler-plate steel. In addition to a relief vent, each tank cell is provided with a pebble-filled separating chamber which permits the escape of gases during normal interrupting duties, condenses oil vapors, and returns the oil in a cooled condition to the tank. The terminal bushings are the usual oil-filled, outdoor type, with sight oil gages and with valves for taking oil samples. The bushings contain the necessary current transformers required for station control.

The disconnecting switches are rated 161 kilovolts, 600 amperes, triple-pole, single-throw. They have pedestal-type insulators mounted in underhung positions. Isolating switches, such as for the oil circuit breakers, are manually operated, and the selective switches for the transfer bus and sectionalizing of the transfer bus are motor operated.

The buses are of 3-inch standard iron pipe size, hard-drawn copper tubing, and are supported on pedestal insulators spaced 32 feet apart. Phase separation is 8 feet 6 inches and ground clearance 7 feet. Connections to the buses are made with 500,000-circular mil bare copper cable and clamp- or bolted-type connectors. The cables are supported with strain insulator assemblies generally, with a few cases where connections are made by copper tubes which in turn are supported by pedestal insulators.

Lightning arresters are installed near the transformers. They are outdoor type, three-phase, suitable for operating on a grounded neutral system. No arresters are connected directly to the buses or transmission lines, but the latter have protective spill gaps where the lines enter the switchyard. The gaps are 36-inch round rings made of 1-inch standard galvanized-steel pipe and are mounted directly on the caps and bases of pedestal-type insulators. The bottom rings of the gaps are furnished with adapters for adjusting the gap spacing from 33 to 45 inches in steps of 1 inch. The transmission lines leaving the structure have overhead ground wires for lightning protection. The ground wires are tied solidly to the steel structure. All steel supporting structures, equipment, frames, and tanks are thoroughly grounded to an interconnected network of bare cables buried in the surfacing of the switchyard. This network in turn is connected by underground cables to the two main grounding mats.

Oil piping consists of one common supply line for transformers and breakers and two separate drain lines, one for transformers and the other for oil circuit breakers, from the oil storage and purifying facilities in the powerhouse to the equipment in the switchyard.

MISCELLANEOUS FEATURES

Galleries, shaft, and operating chamber.

A system of galleries formed in the concrete of the dam is provided for the purpose of access to the operating equipment, experimental equipment, and for maintenance of the drainage system in the dam foundation. An elevator connects all the galleries except the lower, elevation 811. An adit gallery connects the downstream face of the dam with the regulating gate operating gallery, elevation 880. As construction progressed it was found that it would be desirable to have a gallery near the top of the dam. Such a gallery was added to the

design and extends the full length of the dam, exclusive of the spillway section. This gallery conveniently serves as the means of entrance to the head-gate hoists and the headwater float wells. All the porous-tile drains terminate in this gallery, from which they can easily be inspected and cleaned if necessary. All the power and lighting conduits for both ends of the dam are also carried in this gallery. With the addition of the gallery at elevation 1,051 it was possible to raise the entrance adit on the west end of the dam to that elevation.

The gallery at elevation 811 was originally designed for temporary use in drilling and grouting and was to be filled with concrete after it had served its purpose. Subsequent to studies by the field office, a later design omitted the filling and made the gallery permanently serviceable for observation purposes and maintenance work on the drain holes.

The shaft in which the elevator operates is located to the left or east of the spillway. The penthouse which houses the top landing and the operating equipment extends above the roadway. The top landing is at the same elevation as the sidewalks on top of the dam. Three other landings are provided: one at elevation 1,051.50, for access to the gallery leading to the penstock gate operating houses; one at elevation 996.92, for access to the spillway gate operating chambers by way of the gallery at that elevation; and one at elevation 880.35, for access to the adit and to the regulating gate operating chambers.

The elevator is a single-unit installation of the automatic, electrically operated, push-button type. It is designed for alternating-current operation, with a live-load capacity of 3,000 pounds exclusive of the weight of the car and cables. The hoisting machinery is of the 1-to-1 roping single worm gear drive traction type, for a car speed of 250 to 300 feet per minute, with motor, brake, worm gear, and driving sheave mounted on a single self-contained base or bedplate. Brakes of the electrically released, spring-set type with drums keyed to the worm shaft and with actuating mechanism mounted on the hoist bedplate are provided.

The driving motor is 25-horsepower, 2-speed, alternating-current, induction type, with a high internal-resistance, squirrel-cage rotor. Controls are so arranged that the car will start and run on the high-speed winding of the motor and retard and level on the low-speed winding of the motor.

Originally a utility tower was included at the head of the west training wall. This tower was to house a comfort station for the general public and was to balance architecturally the elevator penthouse. It was approximately 24 feet by 15 feet in plan and extended upward from its juncture with the downstream face of the dam to its top at elevation 1,087.25. Since a general utility building had to be erected at the east parking area for the general public, this comfort station was not needed and the entire tower was omitted.

Pumps and sumps.

Leakage into the dam, below elevation 880 is collected in two sumps 3 feet in diameter and 5 feet deep located adjacent to and just downstream from the 811 gallery. One 800-gallon-per-minute and one 22-gallon-per-minute pump for discharging water collected in these sumps are

located in a recess in the downstream wall of the 880 gallery. Water discharged from the pumps together with leakage water into the 880 gallery is drained by gravity through a discharge in the left spillway guide wall and thence into the spillway jump pool.

A sump 3 feet 10 inches square formed in the powerhouse substructure concrete with the bottom at elevation 812 collects most of the powerhouse drainage water. A 300-gallon-per-minute pump and a 6-inch water-jet eductor are provided for pumping this water into the tailrace. A 14-inch eductor is also provided for unwatering the penstock.

Roadway and spillway bridge.

The freeway, which connects State Route 33 at Halls Crossroads with United States Highway 25W at Coal Creek (Lake City), crosses the top of the dam. The top of the abutment section of the dam is widened above elevation 1,053 to provide space for a 22-foot roadway, to sidewalks, and parapet walls. The spillway section is spanned by three concrete-encased through-plate girder bridge spans, supported by the abutment sections and the two piers which divide the spillway into three overflow sections. The parapet walls on the spillway bridge are formed by encasing the plate girders in concrete.

The structural-steel supporting framework for each of the three spans is made up of two plate girders, each 106 feet 6 inches long and 7 feet 6 inches deep at the center of the span, seven 36-inch C. B. 170-pound floor beams, and forty-two 18-inch C. B. 47-pound stringers. The plate girders are spaced 30 feet center to center and joined together by the seven 36-inch floor beams placed 17 feet 6 inches on centers. On one end of each girder fixed bearings are provided, and on the other end the bearings are of the hinged rocker type. Each bridge span is separated from the dam and piers by ample expansion joints to allow movement independent of the dam and piers.

The roadway and bridge are surfaced with Kentucky rock asphalt. The thickness averages about 3 inches at the crown of the road and about 2 inches at the curb. The surfacing was done in accordance with the Tennessee State Highway Department specifications for asphalt paving. Before the asphalt was placed, the low places in the concrete surface were filled with bituminous concrete to smooth the subgrade and insure uniform thickness of asphalt surface.

The bridge floor is drained by means of 3½-inch gutter drains spaced at 35-foot intervals in both gutters of the roadway. The drainage in the abutment-section portion of the roadway is handled by means of cast-iron drains in the gutters spaced on 100-foot centers and draining into 8-inch cast-iron pipes.

DESIGN CHANGES

Had the Norris project been let to contract the entire job would have been delayed pending the completion of plans. Hence it was decided that the best interests of the Authority required that the major construction work be done by force account. This made it feasible to complete the plans as expeditiously as possible and to release them to the field somewhat in advance of construction. The Authority's method of handling construction with its own forces permitted taking advantage of studies and developments completed only

after construction was well under way. One of the chief advantages of the force account method is the flexibility which permits a skilled and experienced construction engineering staff to keep abreast of the job and initiate new or alternate designs in the interests of over-all economy and utility without being obliged to resort to contract revisions which are nearly always unfavorable to the contracting agency in the case of contract work.

Throughout this chapter and in chapters 5 and 6, several notations appear describing various design changes made after construction had begun. These revisions resulted in sizable economies in construction and better selection of equipment or in improvement of the structures with respect either to operating characteristics, general suitability, or aesthetic considerations.

ASSISTANCE FROM UNITED STATES BUREAU OF RECLAMATION

Because Norris was started before an adequate design organization could be recruited and begin functioning in a normal and efficient manner, it was fortunate that the Authority was able to secure the assistance of the United States Bureau of Reclamation in designing its first major project. During the past 35 years the Bureau of Reclamation has participated in the design and construction of some of the largest and most notable dams in the world. These projects have included Boulder, Grand Coulee, Madden, Shoshone, Arrowrock, Elephant Butte, Roosevelt, Cle Elum, and Owyhee, to name but a few. By this arrangement, the design work was started immediately and efficiently without the necessity of delaying several months until a new and competent design organization could be recruited and added to the then rapidly expanding engineering staff of the Authority.

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CHAPTER 4

ACCESS ROADS AND EMPLOYEE HOUSING

Before the construction of the project could actually be started, it was necessary to provide access to the site and to provide housing facilities for many of the employees who were to construct the dam. On many large projects these facilities are temporary and are considered a part of the construction plant, being abandoned or salvaged when the work is finished, along with the other temporary plant needed for actual construction purposes. At Norris, however, the access roads and employee housing facilities for many reasons were of permanent construction.

ACCESS ROADS

Following the location of the dam, one of the first problems to be considered was a suitable means by which equipment, materials, and men could be transported to the dam site so that construction work could be started without delay. The low-type existing roads were narrow, with steep grades and poor alignment. In time of high water parts of the existing roads were actually flooded out. Most of these roads lacked adequate surfacing and could not be used economically for the transportation of all the equipment and material needed for the construction of the dam.

PRELIMINARY CONSIDERATIONS

Type and location.

Two types of access were considered—by highway and by railroad. Access by highway would necessitate the construction of a heavy-duty road from Coal Creek,[1] Tenn., the nearest railroad terminal, to the dam site; and access by railroad would require the construction of a branch line from the nearest and most accessible point on the Southern Railway to the site.

The United States Army Engineers, in reporting [2] on the proposed development of the Clinch River, included a study of the problem of access to the Norris site. Their studies indicated that the construction of a highway in preference to a railroad would be not only more economical but more desirable. Mention was also made in these Army Engineer reports of possible additional access to Knoxville by way of Halls Crossroads.

Preliminary studies by the Authority also indicated that the most economical access would be by highway from Coal Creek, Tenn. These indications were based on both a first-cost consideration and the future use of the highways in the regional development of trans-

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[1] On April 1, 1939, the name of the city of Coal Creek was changed to Lake City.
[2] H. Doc. No. 328, 71st Cong., 2d sess.

portation as compared to the limited useful life of a railroad built solely for construction purposes. Also, a highway would not only serve as a route over which equipment and materials could be transported to the site during construction, but would also serve as a means of access for employees and visitors during and after construction.

In addition to having the site accessible by highway to the railroad terminal at Coal Creek, it was found advisable to have a more direct route to Knoxville, Tenn., where a large portion of the offices of the Authority were located. The necessity for this road was further

FIGURE 70.—*A portion of the Coal Creek-Norris Dam access road. Ditches on the exposed slope controlled erosion until vegetation was started.*

augmented by the construction of the town of Norris, as the distance between Norris and Knoxville was shortened by 6.9 miles when compared to the shortest existing dustproof highway. The best location found for this road started at the site, ran southeast through ridge and valley country for a distance of 16½ miles, and connected with Tennessee State Highway No. 33 at Halls Crossroads, Tenn., about 9 miles northwest of Knoxville.

Design.

After considering a number of highway designs, it was deemed advisable to construct these access highways as freeways, or highways to which there is no vehicular access from abutting properties.[3] The design of a freeway is based on the principle of providing a high-type roadway to take care of the high-speed traffic of today with

[3] Report of Committee on Street Thoroughfares. Transactions of American Society of Civil Engineers, p. 1049. 1935. (In the instance of the Norris freeway it was necessary to modify the freeway principle to the extent of giving a limited number of access permits to landowners whose land had no other highway access.)

maximum safety. It incorporates the best highway practices in respect to easy grades and smooth curves. A freeway-type highway permits a reduction in the number of highway crossings, and therefore decreases the hazards that an unlimited number of intersections would create. It does, however, necessitate the purchase of right-of-way with widths varying from 250 to 350 feet, but the purchasing of this wide strip permits full jurisdiction over the right-of-way and eliminates the possibility of objectionable structures being located adjacent to the highway. Contracts with adjacent property owners protect against exploitation by billboards on private property.

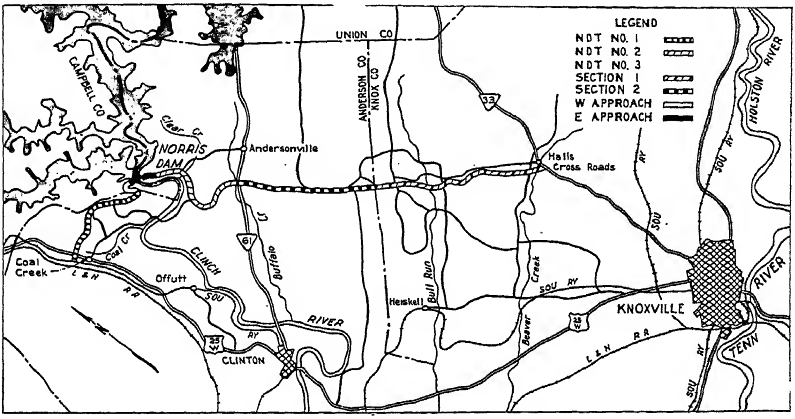

Figure 71.—*Access road location map.*

The general features of the design adopted provided for the location of the road along the natural contour of the ground with a curvature as flat as feasible and with cuts and fills having variable slopes rounded at the top and bottom. Tangents were few and short and none had vertical curves. Adjusted to the topography, this design eliminated insofar as practicable the usual scars caused in road construction. Flat slopes were constructed except in the deep cuts and fills where they were constructed with slopes of 1:2. Another reason for the adoption of this type of construction was the intention to seed and plant shrubbery on the slopes after the road was completed in order to minimize erosion and in general to beautify the highway.

Engineering procedure for final location.

An engineering procedure somewhat different from ordinary practice was used in this work. This method combined the use of stereoscopic studies of aerial photographs and plane table strip topography. The possible general locations were determined by stereoscopic studies of the aerial photographs and by field reconnaissance. Base lines were staked out from these general locations, and plane table surveys were made to secure the topographic features. These maps were made to a scale of 1:600 with 2-foot contour intervals. The

TABLE 29.—*Direct construction costs of Norris freeway*

COAL CREEK TO NORRIS DAM

TVA Force Account Work

COAL CREEK TO CONSTRUCTION OFFICE		
Guard rails, etc.		$6,003.12
WEST APPROACH AND ACCESS ROAD		
Clearing and grading	$20,962.62	
Shoulders	3,995.55	
Fine grading	9,295.84	
Concrete paving	33,188.11	
Concrete structures	17,824.74	
Pipe drains, curbs, and riprap	4,047.75	
Guard rails	5,901.47	
Total		95,216.08

Contract (N. D. T. No. 1)

	Item	Quantity	Unit	Unit price	
1	Clearing and grubbing	24.58	Acre	$200.00	
2	Removal of buildings	Lump sum			
3	Earth excavation, roadway	183,233	Cubic yard	0.3475	
4	Earth excavation, borrow	4,477	Cubic yard	.3375	
5	Earth excavation, structures	2,764	Cubic yard	.50	
6	Rock excavation, roadway	20,660	Cubic yard	.7675	
7	Rock excavation, structures	94	Cubic yard	2.50	
8	Overhaul on excavation	48,794	Cubic yard station.	.0275	
13	Crushed stone base	2,051	Square yard	.68	
15A	Concrete pavement, first lane	27,818.7	Square yard	2.44	
15B	Concrete pavement, second lane	27,618.6	Square yard	2.33	
17C	Concrete, class C	400.75	Cubic yard	17.00	
18	Additional portland cement	6.0	Barrel	2.75	
19A	Reinforcing steel bars	434,423	Pound	.035	
20	Furnishing R. C. pipe	363,795	Pound	.008	
22	Laying R. C. pipe	42,318	Linear feet	.02	
24	Cleaning up	Lump sum			
	Extra work				
	Total				

NORRIS DAM TO HALLS CROSSROADS

TVA Force Account Work

EAST APPROACH ROAD, 400 FEET EAST OF DAM TO POWERHOUSE INTERSECTION		
Clearing and grading 11,262 cubic yards at $0.62	$7,036.19	
Surfacing, 10,595 square yards at $1.09	11,597.89	
Total		$18,634.08
EAST DAM APPROACH, 400 FEET EAST OF DAM, TRAFFIC CIRCLE, AND POWERHOUSE ROAD		
Grading, 25,360 cubic yards at $0.62	$15,844.22	
Surfacing, 14,159 square yards at $1.09	15,499.24	
Total		31,343.46
HALLS CROSSROADS TO INTERSECTION OF POWERHOUSE ROAD		
Clearing, grading, and drainage	$174,073.41	
Surfacing	288,050.13	
Structures	80,432.86	
Guard rails	5,796.43	
Total		548,352.83
Total direct construction cost of TVA work		699,549.57
Total cost of project		

Contract (N. D. T. No. 3, Sec. 1)

	Item	Quantity	Unit	Unit price	
1	Clearing and grubbing	9.6	Acre	$100.00	
2	Removal of buildings and structures	Lump sum			
3	Unclassified excavation, roadway	204,891	Cubic yard	0.425	
3A	Unclassified excavation, channel change	6,445	Cubic yard	1.00	
4	Excavation, borrow	10,994	Cubic yard	.40	
5	Earth excavation, structures	2,000.6	Cubic yard	1.00	
7	Rock excavation, structures	219.1	Cubic yard	7.50	
8	Overhaul excavation	266,626	Cubic yard station.	.02	
17C	Class C concrete	750.45	Cubic yard	19.00	
19	Reinforcing steel	43,176	Pound	.04	
20A	12-inch reinforced concrete pipe [1]	435	Linear feet	1.00	
20B	15-inch reinforced concrete pipe [1]	1,596	Linear feet	1.35	
20C	18-inch reinforced concrete pipe [1]	297	Linear feet	2.00	
20D	24-inch reinforced concrete pipe [1]	378	Linear feet	2.50	
20E	30-inch reinforced concrete pipe [1]	174	Linear feet	4.20	
20F	36-inch reinforced concrete pipe [1]	99	Linear feet	6.00	
24	8-inch vitrified clay pipe underdrain	117	Linear feet	.50	
28	Cleaning up	Lump sum			
	Extra work				
	Total				

Contract (N. D. T. No. 3, Sec. 2)

	Item	Quantity	Unit	Unit price	
1	Clearing and grubbing	10.44	Acre	$150.00	
2	Removal of buildings and structures	Lump sum			
3	Unclassified excavation, roadway	145,687	Cubic yard	.44	
3A	Unclassified excavation, channel change	2,754.4	Cubic yard	.50	
4	Excavation, borrow	4,073	Cubic yard	.44	
5	Earth excavation, structures	1,828.6	Cubic yard	1.00	
7	Rock excavation, structures	410.9	Cubic yard	3.00	
8	Overhaul on excavation	89,248	Cubic yard station.	.02	
17C	Class C concrete	563.0	Cubic yard	20.00	
19	Reinforcing steel	33,810.3	Pound	.04	
20A	12-inch reinforced concrete pipe	231	Linear feet	1.00	
20B	15-inch reinforced concrete pipe	1,299	Linear feet	1.25	
20C	18-inch reinforced concrete pipe	132	Linear feet	1.50	
23F	36-inch corrugated galvanized metal pipe	94	Linear feet	5.45	
23G	42-inch corrugated galvanized metal pipe	802	Linear feet	6.35	
23H	54-inch corrugated galvanized metal pipe	130	Linear feet	8.20	
24	8-inch vitrified clay pipe underdrain	258	Linear feet	.60	
28	Cleaning up	Lump sum			
	Total				
	Total cost of contract work				

[1] Furnished and in place.
[2] This figure agrees with total of contract and force account construction as shown on page 172 (see note 1).

Amount earned
$4,916.00
3,000.00
63,673.47
1,510.99
1,382.00
15,856.55
235.00
1,341.84
1,394.68
67,877.63
64,351.34
6,812.75
16.50
15,204.80
2,910.36
846.36
1,000.00
1,971.39
254,301.66
$960.00
1,800.00
87,078.68
6,445.00
4,397.60
2,000.60
1,643.25
5,332.52
14,258.55

completed plane table sheets were used to prepare the location on paper in the office. The paper location was then staked out and reviewed in the field and, after minor adjustments, was computed and restaked.

CONSTRUCTION

For purposes of construction the entire freeway from Coal Creek to the dam site and from the dam site to Tennessee State Highway No. 33 at Halls Crossroads was divided into four parts (see fig. 71).

The portion of the access road most urgently needed for the transportation of equipment and materials to the dam site was that section from the railhead at Coal Creek to the dam site. In order to speed construction of this section, fills were compacted in 6-inch layers, allowing paving immediately to follow the grading, and one complete lane was paved first so that hauling could begin while the second lane was being completed.

When construction plans for the section from Coal Creek to the site (N. D. T. No. 1) were completed, studies were begun for the remaining freeway sections (N. D. T. No. 2 and N. D. T. No. 3, secs. 1 and 2). As soon as the location was definitely established and enough plans completed to start construction, work was begun. To expedite the construction and make the whole project available for use as soon as possible, the grading and drainage of sections 1 and 2 of N. D. T. No. 3 were contracted as individual projects. The Authority's forces did the drainage and grading work for N. D. T. No. 2 and the bridge construction and paving for all sections excepting N. D. T. No. 1. This assignment of work allowed construction to proceed on several sections simultaneously. One reason for performing portions of the work by the Authority's forces was that certain portions of the plans could be released in advance, thus allowing the purchase of materials while the remainder of the plans were being completed. It was found also that the plans for grading and drainage for N. D. T. No. 2 could be prepared from the topographic sheets without additional cross sections. This permitted work to begin at once on that section. By the time the Authority's forces had completed the grading and drainage work for N. D. T. No. 2, the contractors had completed the grading and drainage for N. D. T. No. 3, sections 1 and 2. With this work completed, a crushed-stone base and a mixed-in-place bituminous surface were laid for the length of road from the east abutment of the dam to Halls Crossroads by the Authority's forces; that portion of the freeway from the west abutment to Coal Creek had previously been concreted by contract.

Miscellaneous temporary construction plant roads were built by the dam construction forces to connect the access roads to the various parts of the construction plant and dam as needed.

When the dam was completed, a west approach road 1.15 miles in length and an east approach road 0.2 mile in length were built and connected by a roadway constructed on top of the dam.

Coal Creek to dam site section.

Construction of the section of the access highway from Coal Creek to the dam site, with the exception of a small amount of miscellaneous work done by the Authority, was done under contract by W. W. Boxley & Co. of Roanoke, Va. Bids were opened on October 27, 1933, notice

of award of contract was given on November 1, 1933, and work was begun on November 2, 1933. The first lane was completed and opened to traffic on January 23, 1934. All paving under this contract was completed on February 15, 1934, after only 69 working days. The work was accepted on March 26, 1934. A tabulation of bids received for this work is shown in table 30.

TABLE 30.—*Tabulation of bids for construction of access road between Coal Creek and the dam*

Bids	Bids as submitted	Bids adjusted [1] for comparison	Bids	Bids as submitted	Bids adjusted [1] for comparison
1	$256, 223. 50	$256, 223. 50	6	$303, 125. 00	$309, 125. 00
2	276, 955. 00	279, 155. 00	8	305, 070. 00	311, 070. 00
3	282, 500. 00	284, 700. 00	7	310, 020. 00	316, 020. 00
4	295, 640. 00	295, 640. 00	9	333, 860. 00	401, 060. 00
5	301. 550. 00	302, 750. 00	10	353, 890. 00	406, 090. 00

[1] Bids adjusted to include payment of penalty provided in specifications for additional time to complete work.

The wearing surface is a heavily-reinforced concrete pavement 22 feet wide. Large pieces of transportation equipment which were to use the road during the dam construction period made this width

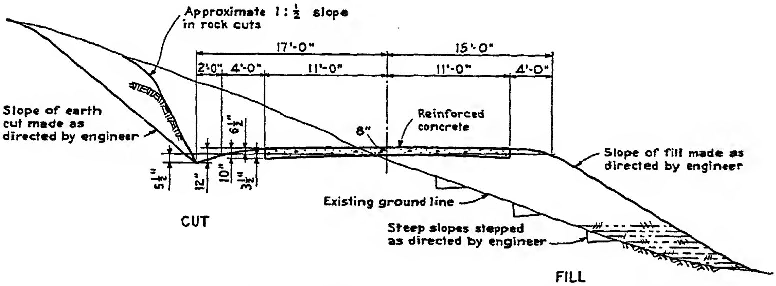

FIGURE 72.—*Typical cross section of the concrete heavy-duty access road.*

necessary. Expansion and contraction joints were provided every 60 feet in addition to a longitudinal center joint. All drainage structures were built either of reinforced concrete pipe culverts with concrete headwalls or reinforced concrete box culverts. Guard rail of the metal-plate type was provided on all high fills and on sharp curves.

The principal quantities involved in the construction of this road were:

Length	miles	4. 31
Maximum gradient	percent	7. 8
Maximum curve—radius	feet	298
Earth excavation	cubic yards	190, 474
Rock excavation	do	20, 754
Concrete paving	square yards	55, 437
Concrete, class C	cubic yards	401
Steel reinforcing bars	pounds	434, 423

The total contract amounted to $254,301.66. Table 29 shows the final estimate quantities and unit costs for this contract.

Dam site to Halls Crossroads section.

The construction of this section of roadway was done in three separate parts, that portion between Norris Dam and Tennessee State Highway No. 61 being built entirely by the Authority's forces. The drainage and grading work for the portion from Route 61 to Halls Crossroads was contracted in two sections. Paving and bridge building for these two sections were done by the Authority. Right-of-way for this highway in Knox County was furnished by the county; the remainder was purchased by the Authority.

The pavement for this section is 20 feet wide, consisting of a 6-inch compacted, water bound, crushed stone base and a mixed-in-place bituminous surface. Grades and curves were so located that the road could in the future be given a higher-type pavement should it become part of the State or Federal highway system and the traffic become heavy enough to justify this betterment.

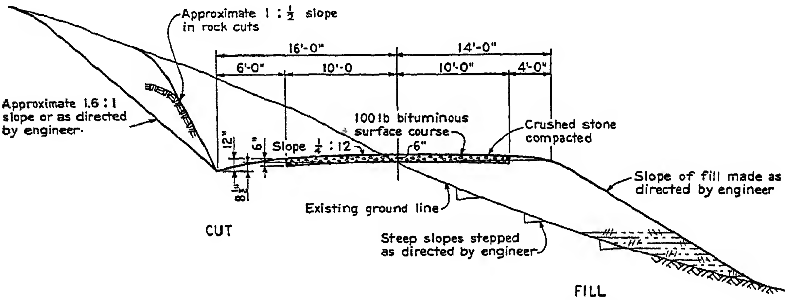

FIGURE 73.—*Typical cross section of access road, Norris Dam-Halls Crossroads.*

TABLE 31.—*Principal features of N. D. T. Nos. 1, 2, and 3*

		N. D. T. No. 1	N. D. T. No. 2	N. D. T. No. 3 Sec. 1	N. D. T. No. 3 Sec. 2
Length	miles	4.31	4.86		4.92
Maximum grade	percent	7.80	7.40	9.879	7.611
Maximum curve	radius—feet	298.00	317.18	500.00	576.22

Several drainage structures were necessary on this section of road. For the smaller streams reinforced-concrete pipe culverts, reinforced-concrete box culverts, and pipe culverts of the paved invert corrugated metal type were used. Over Clear Creek a reinforced-concrete rigid frame structure with a 25-foot span capable of supporting an earth fill was used. To cross Bull Run Creek a 130-foot, 3-span, continuous reinforced-concrete T-beam girder-type bridge was constructed. A 61-foot 8-inch reinforced-concrete rigid frame of the through-girder type was used to span Hinds Creek, and a 25-foot reinforced concrete barrel arch with earth fill spandrels was used to span Buffalo Creek.

The contract for grading and drainage structures of the Halls Crossroads-Knox County line portion (N. D. T. No. 3, sec. 1) was awarded February 27, 1934, to Chandler Bros., Inc., of Virgilina, Va. A tabulation of bids received for this work is shown in table 32.

TABLE 32.—*Tabulation of bids for construction of access road between Halls Crossroads and the Knox County line*

Bids	Total of bids including paving	Total of bids without paving	Bids	Total of bids including paving	Total of bids without paving
1	$193, 994. 10	$132, 394. 10	5	$224, 138. 60	$158, 688. 60
2	195, 205. 75	141, 305. 75	6	232, 178. 65	166, 728. 65
3	216, 890. 77	149, 130. 77	7	278, 634. 50	209, 334. 50
4	225, 830. 40	156, 530. 40			

Work was started immediately after the award was made and completed on June 30, 1934. The principal quantities of construction were:

Excavation, unclassified	211,336 cubic yards.
Concrete	750 cubic yards.
Reinforcing steel	43,176 pounds.

The total contract amounted to $132,349.86. The final estimate quantities and unit costs for this contract are shown in table 29.

The contract for the drainage structures and grading for the portion of roadway between the Knox County line and Route 61 (N. D. T. No. 3, sec. 2) was awarded to Young and Lyons, Rogersville, Tenn., on February 26, 1934. The bids received for this work are tabulated in table 33.

TABLE 33.—*Tabulation of bids for construction of access road between the Knox County line and Route 61*

Bids	Bids as submitted	Bids adjusted for comparison	Bids	Bids as submitted	Bids adjusted for comparison
1	$96, 352. 15	$97, 652. 15	4	$110, 281. 25	$110, 281. 25
2	103, 345. 75	103, 345. 75	5	120, 327. 00	120, 327. 00
3	107, 055. 25	107, 055. 25	6	124, 538. 00	124, 538. 00

Work was begun on March 7, 1934, and completed on June 15, 1934. The principal quantities of construction were:

Excavation, unclassified	148,441 cubic yards.
Concrete	563 cubic yards.
Reinforcing steel	33,810 pounds.

The cost of this contract was $99,974.84 which includes $1,300.00 paid to the contractor as a bonus for completing the work 13 days prior to the contracted date of completion. Final estimate quantities and unit costs are shown in table 29.

Grading on the portion of freeway from Route 61 to the dam site was started by the Authority on February 15, 1934. The length of

this portion is 4.86 miles. The principal quantities in construction were approximately:

Excavation, unclassified	151,000 cubic yards.
Concrete	140 cubic yards.

Surfacing of the entire length of 16.24 miles from Halls Crossroads to the intersection with the powerhouse road, the construction of all bridges for this section, the erection of the guard rails, and a small amount of miscellaneous grading and clean-up were done by the Authority. Surfacing was started as soon as possible after the grading work was completed and was finished before July 15, 1934.

The quantities involved in the construction of the bridges and the placing of the surfacing are shown in table 34.

TABLE 34.—*Construction quantities*

Structures	Excavation	Concrete	Reinforcing steel
	Cubic yards	*Cubic yards*	*Pounds*
Bull Run Bridge	855	672	97,126.5
Hinds Creek Bridge	287	326	67,832.0
Buffalo Creek culvert	299	238	25,832.0
Clear Creek culvert	305	428	55,812.0
Total quantities	1,746	1,664	246,602.5

Macadam surfacing (approximate):

	Cu. yd.
6-inch macadam surface	134,200
Mixed-in-place bituminous surface	7,365

The total direct construction cost of all the work done by the Authority for this section of road was $548,352.83.

Approach roads.

When the construction of the dam was completed, east and west approach roads were required to connect the two access highways in order to complete the freeway from Halls Crossroads to Coal Creek. On the east abutment a road 0.2 mile in length of the same type as the highway from Halls Crossroads to the site was constructed to form the approach on that side. Similarly, on the west abutment a road 1.15 miles in length of the same type as the road from Coal Creek to the site was constructed to form the approach on the west side. These approach roads were connected by the roadway over the dam.

Costs.

Norris freeway provided access to both the dam and camp sites throughout the construction period. By agreement between the Authority and the State of Tennessee, title to the freeway land outside the reservation limits was deeded in fee, and within the reservation easements were granted over the right-of-way for the purposes of roadway maintenance. The roadway from Tennessee Highway 33 to United States Highway 25W, with certain exceptions, was also transferred to the State of Tennessee.

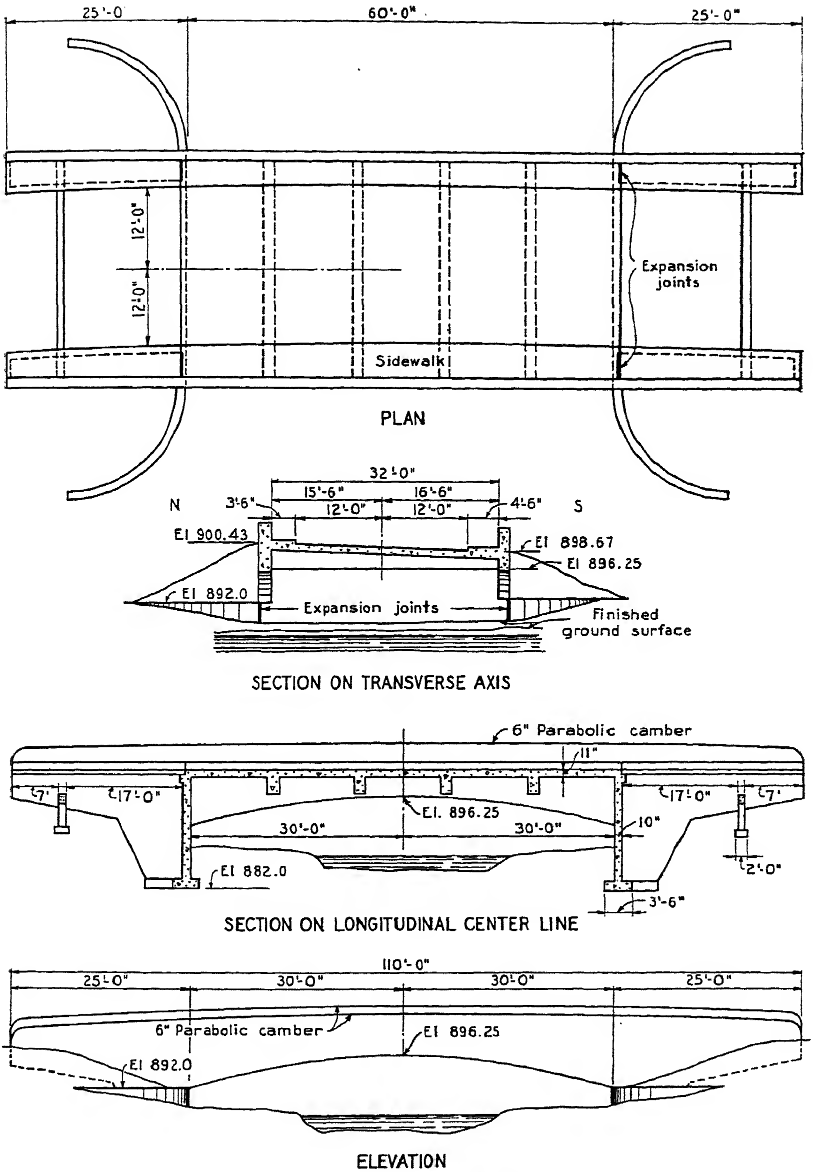

Figure 74.—*Hinds Creek Bridge.*

The allocation of Norris freeway costs has been made as shown in table 35. The total cost of the freeway, $1,454,465.17, includes in addition to the direct construction cost (table 29) the expense involved in land acquisition, indirect construction cost, and distributive general expense. The allocation of the freeway cost to general plant, indirect construction cost, and hydraulic multiple purpose plant, is shown in more detail in tables 100–B, 100–C, and 101.

TABLE 35.—*Allocation of freeway costs*

	Total cost of freeway	Allocated to—		
		General plant	Indirect construction costs	Hydraulic multiple purpose plant
Land and acquisition expense	$22,946.83	$14,328.24	$8,618.59	
Direct construction costs:				
Contract work [1]	485,326.34	243,488.25	241,838.09	
TVA force account [1]	699,549.57	379,617.64	270,763.73	$49,168.20
Indirect construction costs	[2] 76,315.24	35,497.07	32,807.19	[2] 8,010.98
Distributive general expense:				
Design and construction engineering costs	127,679.50	69,504.62	53,847.24	4,327.64
Executive and administrative costs	31,592.86	15,973.53	12,819.90	2,799.43
Other general costs	11,054.83	10,175.43		879.40
	[2] 1,454,465.17	768,584.78	620,694.74	[2] 65,185.65

The costs allocated above cover the following sections of the Norris freeway:

	Title rests in—
General plant:	
United States Highway 25W to Coal Creek yard	U. S. Government.
Tennessee Highway 33 to West Norris Road	State of Tennessee.[3]
Dam indirect construction costs:	
United States Highway 25W to 126 feet west of dam	Do.[3]
400 feet east of dam to West Norris Road	Do.[3]
Hydraulic multiple purpose plant:	
126 feet west of dam to end of dam	U. S. Government.
East end of dam to a point 400 feet east of dam including traffic circle and powerhouse road	Do.

[1] A detailed analysis of these 2 items appears in table 29.
[2] Includes $2,381.39 distributed from indirect construction costs.
[3] Excludes land within reservation limits.

EMPLOYEE HOUSING

Contrary to standard practice for building construction camps to house workmen, it was decided to construct for this project a permanent town instead of a temporary camp. Under typical construction practice, a dozen or more houses of excellent quality would have been built for construction executives, a compact group of bunk houses of temporary construction would have been located near the dam site for unmarried workmen, and space would have been allotted for the construction of huts by workmen who desired to bring their families to the site. Instead of a temporary camp, the establishment of a new town was deemed feasible because of the permanent character of the Authority's operations in the headwater region of the Tennessee River, the need for a secondary administrative center in these regions, and because of the housing requirements for the permanent operating force of the dam.

GENERAL

The United States Army Engineers' plan[4] for this project included a layout for a construction camp to house 2,500 employees at an estimated cost of $1,434,300. Such a camp involved the construction of roads and sewerage and water-supply systems which would have been of approximately the same value as those that would be necessary for a permanent community. In view of the investment that would be necessary in utilities and buildings, it was thought that it would be worth while to go a step further and make the housing of good enough quality to have a normal, useful life instead of building temporary structures. In view of these considerations, a decision was reached to expend more for the town than was contemplated in the Army estimate and to aim at an important residual value after the community's term of service as a construction camp was ended.

The original plan for the village coincided with a desire on the part of the Authority to encourage by means of demonstration a combination of industry and agriculture. It was thought that in spite of the lack of rail facilities the abundant supply of electric power at Norris would facilitate the establishment of several industries whose employees could supplement their earnings by subsistence farming. In addition to the proposed combination of industry and agriculture, the trend of thought in the Authority favored extensive employee-training activities.[5]

Early studies.

A small village is not a good economic unit, and for this reason it was decided to provide for a possible ultimate capacity of about

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[4] H. Doc. No. 328, 71st Cong., 2d sess.

[5] Dawson, J. Dudley, Progress in the Tennessee Valley. The Federal Employee, November 1934, p. 7.

5,000 persons. Discussions began with a program of about 1,000 houses in mind. This figure in October 1933 was reduced to about 500, and soon afterwards the initial development was cut to about 250 houses, exclusive of the dormitories. This was subjected to further modification and the living facilities of the village as it now stands consist of 294 single houses, 10 duplex houses (20 families), and 5 apartment buildings (30 units).

While the project had been contemplated by the United States Army Engineers for a long time, and while the dam was being designed largely by the Bureau of Reclamation, the design of a permanent community such as the one built was unprecedented and was the first TVA planning for actual construction. The construction schedule for the town was of necessity linked very closely with that of the dam, since it was to house employees engaged in dam construction. During the preliminary studies the entire basis for estimating labor was shifted from the low-wage scale then prevailing in the neighborhood to a relatively high-wage scale comparable to that normally paid in Government construction work. During the period of rapidly changing conditions in the fall of 1933 and winter of 1933-34, a further complication arose from the establishment of the N. R. A. which, while protecting basic industries, precluded the buying of material at bargain prices.

The Authority's town planning staff was encouraged to approach the design problem from a fresh point of view and to make a complete reappraisal of essential and nonessential points in housing and town planning. Such an undertaking in a limited time presented many difficulties but nevertheless presented the rare advantage of conceiving a complete development of the camp and town as a creative problem.

The two precedents that were most in the minds of the designers at this time were Kingsport, Tenn., and Radburn, N. J., both of which were designed as model communities, but under different circumstances. In reviewing Kingsport, the need was felt for protecting the new community by a belt of publicly owned land to prevent undesirable encroachment. Therefore, to insure for the community the maximum residual value after its temporary use as a construction camp was over, a protective belt of Government-owned land was designed, both to preserve the favorable sociological conditions that were created in the construction camp, and to protect property values in the permanent town. At construction camps where no precautions are taken, injurious developments generally take place nearby, and the same thing will happen in towns which are carefully planned in themselves but which have no precaution against encroachment on the outskirts.

In reviewing the plan of Radburn, which represented the latest development of its kind in the United States, it became apparent that the type of street layout which had gained favorable attention could not be applied in the case of Norris on account of topographic conditions; and the town plan had to be strictly related to the peculiarities of the ground.

In approaching the question of architecture, the planners aimed at rationalized designing, tempered, however, by sympathetic treatment in basic harmony with the architecture of the region. Throughout the

design activities there was an earnest endeavor to preserve the natural advantages of the surroundings.

The design of the camp buildings was begun slightly later than the study of housing and had to mature rapidly since the camp was to be built first. The roads, sewerage, and water supply systems had to be laid out to serve both the village and the camp and to conform to topographic limitations of the site.

In approaching the design problem, the Authority sought and obtained advice from the United States Department of Agriculture, the University of Tennessee, Anderson County, and other Governmental agencies. To assist in the design of the construction camp and village, four consultants of national reputation, including an outstanding local architect familiar with architectural requirements of the region, were retained. The work also involved close cooperation with the Civilian Conservation Corps and with the National Park Service. The task of coordinating all the lines of thought and action was a difficult one to carry on in conjunction with the rush requirements in the design of the new community. During the progress of design, the construction schedule that had been contemplated for the dam was speeded up so that the construction of the town became even more urgent than was at first expected.

It was obvious that the camp and village were to combine the purposes of employee housing and training activities along with experimental development and demonstration of improved techniques in several fields. It was not clear what relative weight should be attached to each of these objectives, and as the program went on there were radical changes of objectives because more complete information was secured and new ideas were contributed. The broad purposes of the Authority were to establish a wholesome environment through plain, direct means, eliminating all that was superfluous and letting the natural environment count at its maximum value.

One of the major changes that took place during the course of design in relation to housing occurred when it was decided to make the first group of 150 houses serve as a demonstration of electric heating and electrical equipment. The opportunity to test and demonstrate the possibilities of electric heating was considered to have a substantial cash value by the Authority. In computing to what extent this improvement was worth while, the extra cost of the installation was taken into account; but possibly it was not recognized immediately that the improvement carried other implications; namely, the families who were to use electric heating and electric kitchen equipment would necessarily have to be people who could pay a substantial rent. It became apparent that a different kind of occupancy must be anticipated than the minimum income, rural type of occupancy that was originally contemplated. This in turn meant a moderate upward revision of the quality of interior finish and appointments in the house to a standard which, while remaining plain, would be acceptable to families of medium income.

Various other important changes were introduced in April 1934 in connection with the types of houses, development of added space, and other details. As it became apparent that both the quality and the unit costs of construction (wages and materials) were shifting, the number of houses was reduced. The second group of houses was designed on an entirely different basis, reverting to the purpose of con-

structing houses at the least cost consistent with a reasonable degree of permanence and homelike conditions. As the construction camp proper was designed and built rapidly, and as fewer problematic factors were involved, there were fewer changes of objective.

Preliminary surveys for the town planning and architecture were started in August 1933, working drawings for houses were started in November 1933, and regular construction of houses was begun in January 1934. Many building sites at that time were inaccessible by road, and planning work was only partly completed. However, most of the first 150 houses were in full use by September 1934, although final details of electrical equipment on these houses were delayed until some weeks later. The designing and building of the construction camp preceded the houses, and the camp was in use by January 1, 1934. The second group of 80 houses was designed in the spring of 1934 and completed by the fall of 1934. The five apartment-house buildings were built about the same time. In the summer of 1934, 49 additional single houses and 10 duplex houses were designed. These were built during the following winter. The town was essentially complete in its present form, including utilities, roads, and civic buildings, in the spring of 1935.

Relation of Norris to development of national housing policy.

During the design of Norris, the question of a national policy toward housing was beginning to come to the foreground; but the problem had not been explored by actual trial in the United States except under emergency of the World War. It had long been a subject of active study and experimentation in other countries, various developments in England being considered the most applicable in this case.

The work as carried on in the building of Norris was an example of public procedure in planning in the sense that it was not a personal expression of one or more individuals, but was a composite of the thought of many persons. It was subjected to many checks and balances which were characteristic of public opinion.

Since it preceded other efforts (P. W. A., F. H. A., Resettlement Administration) of a similar nature in the United States, it was exploratory in many respects.

Significance of Norris in town planning.

If for no reason other than the rarity of planned communities in the United States, Norris is significant. Every large city has its fine residential suburbs in which homes compete with each other on a very substantial basis of expenditure. Such developments are so frequent and so far removed from the major problem of housing in America that they attract little interest, no matter how high their quality. The peculiarity of public planning as undertaken at Norris is that it aims at achieving an essentially fine environment through plain means and minimum expenditures. Any effort in this direction is eagerly scrutinized to see what steps forward have been taken in a task many publicists consider a major social and economic problem of the Nation at this time.

Most public housing enterprises are slum clearance projects or other urban developments. Norris contributes a rather unique aspect of public housing in that it contemplates living facilities situated well

outside a city, a conception which is beginning to rival slum clearance in the minds of housing authorities as a correct solution of social and economic problems. The design of Norris is a sober attempt to visualize the kind of living conditions that the American people as a whole may legitimately aspire to have if productive energies are utilized to the best advantage.

The design of Norris provided for preservation of a natural and interesting environment which was acquired, and of a kind that can generally be acquired, in its unimproved state at very moderate cost. In this environment nothing pretentious was erected, but everything ugly and superfluous was excluded. There is an expression of unified development without monotony, and there is an intimate relation of each home to rural surroundings. At the same time, nothing is lacking in the matter of streets and utilities to give the advantages of living in a larger town, for all the public services are fully developed. An atmosphere of simple, wholesome, and very convenient living is established which has a potent effect on the general spirit of the town.

A major contribution to health and cleanliness is demonstrated in electric heating. The advantages of this controlled environment are adequately preserved by the protective belt which guards against undesirable developments around the town. In the average community the pride that is felt in the best quarters of the town is offset by the sordid conditions in other parts. While no part of Norris is extravagantly fine, the net result of a community which is clean and well considered in all its parts is sufficiently striking to commend it as a demonstration.

This significance has not been overlooked by other private and public agencies. Representatives of subsistence homesteads, F. H. A., P. W. A., the Resettlement Administration, and many others, including representatives of foreign governments have made official inspections. Principles used in the design of Norris have since been applied elsewhere. One magazine [6] has stated that "Of the several Government housing developments, the TVA project at Norris, Tenn., has probably contributed more than any other to the basic study of the technique of small house design."

Permanent value of Norris.

The character of construction at Norris is reasonably permanent. While the work is not highly finished, it is sound and thorough, designed for low maintenance cost. For example, front entrance stoops of most of the houses are built of brick, stone, or concrete.

Minor changes in assumptions on which the design work was based caused some inconsistencies in both architecture and town planning. However, in the main, the principles and ideas first evolved were put into effect; and Norris provides a good example of numerous advantages gained by unified handling of a housing project.

Not the least advantage is the principle of protecting property values against excessive depreciation and obsolescence by controlled development under one management. This principle is now being recognized as a very significant factor in relation to both private and public property.

[6] Federal Home Owners Loan Review, pp. 408, 409, September 1937.

The present developed portion of the town is mostly on one side of the community center, but provision has been made for future expansion on the opposite side, and some of the most desirable building sites are in that area. Partial provision has been made in the sewer and water system to take care of such a future expansion, which would bring the town more nearly to what is generally considered an effective size as an economic unit. The probability of industries being established at Norris cannot be estimated at the present time. It is certain that recreational developments and tourists' interest in Norris Dam and Lake will affect advantageously the economy and development of the community. The demand for living facilities at Norris is somewhat greater than was anticipated, and as long as some of the TVA activities are more or less centered in the Norris area, a combination of employee housing service activity, and satellite community residential factors will insure usefulness of the town.

Proposed and actual value of Norris.

A successful unity of construction camp and village was achieved. The occupancy of both the dormitories and the houses was by higher classifications of employees than originally contemplated. The shift of emphasis was due partly to the shift toward a permanent type of housing, but the difficulty of housing low-income employees on an economic basis was inherent and had to be deliberately recognized if the lower classifications were to be housed.

Besides establishing a unified community spirit by combining the camp and village, there was a successful effort to serve both the rural people who came in from surrounding regions to work on the dam and the employees of far different origin and habits who came from all parts of the United States.

With the conclusion of the construction of the dam, a transition took place to a more permanent type of occupancy. In this phase, Norris functions mainly as a satellite community to Knoxville. Rental values have gradually increased for most of the types of permanent houses, and the school and other municipal facilities are recognized as being of high quality.

DEVELOPMENT OF SITE

Site requirements.

It was determined by the board of directors that the facilities provided should consist of a construction camp housing about 1,000 men and a village large enough to accommodate 250 families, but on a site suitable for a community of 1,000 families. Included in the basic assumption made in connection with this housing program were the following:

1. The site must be not more than 3 miles from the dam site.
2. It must be scenically attractive.
3. It must lend itself to reasonably economical development.
4. It should be on land as good for farming and gardening as the other limitations would permit.
5. It should include enough area so that an adequate "protective belt" would guarantee the new community freedom from encroachment by shack development and nuisance areas.

Specifically this meant that about 2,000 acres must be found within 3 miles of the dam site which would include a 300-acre tract suitable for a village, and approximately 1,000 acres of reasonably good farm land.

Table 36 gives the amount of land actually used for residential and other areas in the present development and estimates for possible future developments.

TABLE 36.—*Amount of land used and amount available*

	Residence		Other		Total	
	Acres	*Percent*	*Acres*	*Percent*	*Acres*	*Percent*
Present development	129.3	6.6	[1] 110.7	5.6	240.0	12.2
Future extensions	234.0	11.9	[1] 151.0	7.7	385.0	19.6
Protective belt			1,333.0	68.2	1,333.0	68.2
Total	363.3	18.5	1,594.7	81.5	1,958.0	100.0

[1] Commercial area occupies 1.5 acres, the remainder is developed park area.

1. Families per acre of residence area:	
Present development	2.7
Total development	3.2
	Acres
2. Total area in town proper	1,958
Total area in Norris Park	3,887
Total	5,845

Location procedure.

Aerial photographs taken in 1929 covering part of the area in question were available, and from these a base map showing existing culture to a scale of 1:7,200 and contours at 50-foot intervals was prepared. The topographic information was useful in giving a general picture of the region, but it was not accurate enough for more specific use. A property map to a scale of 1:15,000 compiled from aerial photographs and other sources, the United States Geological Survey sheets, Geological Survey maps, and county highway maps, were also available.

Preliminary office study narrowed the feasible area down to a strip of land 2 miles wide and 3 miles long. It then became necessary to make a rapid but intensive study of this property in order to determine which portion of it was best suited to the contemplated project. Almost at once it became obvious that the problem was too complex to solve by field inspection or by studying the available maps. More detailed information was needed, especially with respect to topography, rock outcrop, and quality of soil.

It was essential that the village and camp site be selected as soon as possible because existing housing accommodations in the area were scarce. As additional information was urgently needed, it was assembled as rapidly as possible.

A survey to a scale of 1:2,400 was made showing the location of rock outcrop, buildings, fences, roads, and other observable features, and indicated topography with 5-foot contour intervals. The survey parties also made notes on the quality of the soil. This

survey was instrumental in determining the 300-acre tract most suitable for town-site development. It did not, however, serve as a basis for detailed site studies as the topography was not of sufficient accuracy. It was necessary to resurvey to a scale of 1: 600, showing topography in 1-foot contour intervals, the 300 acres chosen as the town site. With the detailed topography available, site planning proceeded rapidly.

The first preliminary study for the lay-out of Norris which was concrete enough to be acceptable covered very closely the area now occupied by the town. It indicated a freeway bypassing the town

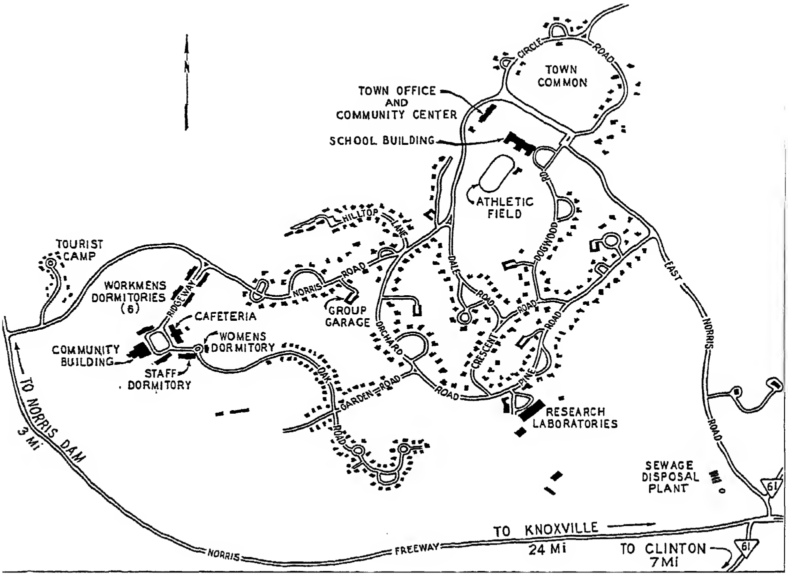


FIGURE 75.—*Site lay-out.*

and a protective belt surrounding the developed community. The locations for the construction camp in the western part of the village and the village center were established by this study and were changed but little later on. The village lay-out featured cul-de-sacs, or small isolated house groups, and scattered apartment groups, both of which were modified as plans were developed. Figure 75 shows the village as it was finally laid out and built.

The study based on the 1:2,400 scale topography proved to be impractical in detail. A lay-out on the larger scale topography (1: 600) resulted in placing most of the streets in the valleys in order to avoid grades steeper than 8 or 10 percent and in order to leave desirable house sites free for development. House sites on hillsides and on ridges were selected, and a great deal of attention was given to avoiding slopes greater than 20 percent in order to avoid high

construction costs. An open, rambling, informal sort of lay-out resulted because of the rough topography, which had little in common with the highly organized schemes developed for similar projects in England and at Radburn in this country.

The utility systems were less than normally related to the street lay-out because of the rough topography. The final development of the plan produced an informal rural village, somewhat inconvenient as to access from some dwellings to the street system, and, because of unavoidable topographic limitation, somewhat high in site-development cost. The general lay-out together with the utilities serving it, was designed with a thought of possible future expansion. As a consequence, a community of 1,000 families can be accommodated at the site with comparatively few adjustments in the present plant. New streets and extensions of utilities will be required, but plant capacity can be increased at little cost. About 82 house sites remain unoccupied in the present developed area, which now accommodates 344 families. Table 37 gives the housing statistics for present and possible future developments.

TABLE 37.—*Housing statistics for present and possible future developments*

Type of dwelling	Present developed area						Final development—new streets and sewer system	
	Existing sewer system				With short extension on existing sewer system			
	Existing		Possible					
	Buildings	Families	Buildings	Families	Buildings	Families	Buildings	Families
1-family	294	294	325	325	376	376	1,098	1,098
2-family	10	20	10	20	10	20	10	20
Multifamily	5	30	5	30	5	30	5	30
Total	309	344	340	375	391	426	1,113	1,148

UTILITY SYSTEMS

Streets, public areas, and paths.

The design of the streets is based directly on the topography of the town site. Special attention was given to providing easy grades, alignments, and frontage along the streets that would be practical for building sites. West Norris Road and East Norris Road make up the main artery for traffic passing through the town. This road is wider and of heavier construction than the other streets. The width of pavement in the various streets varies from 16 to 20 feet. The total length of paved streets is 32,234 linear feet. There are 64,913 square yards of paved surface consisting of macadam, generally 6 inches thick, penetrated with asphalt, and finished with a topping of screening and asphalt (80-pound treatment per square yard).

Gutters were developed and paved little by little after the town was completed. Parking inserts have been provided at intervals on most of the streets. To facilitate crossing streets in safety, three pedestrian underpasses were built at points where heavy traffic might eventually develop. The topography permitted these to be located

under bridges. Large parking spaces were provided near the construction-camp buildings and other points of public assembly.

In order to preserve the informal aspect of the town, sidewalks were provided generally only on one side of the street and not necessarily parallel to or very close to the street. The paths run freely in such a way as to avoid destroying good trees and to strike a pleasant course without clinging to the edge of the pavement. Additional paths cross the interior of the blocks. These were laid out to provide safe, direct routes to and from the school and the community center.

Water-supply system.

The water-supply system for the camp and village is of conventional design. The population figures used as basis for the design of the system were 2,000 for the present town and 4,000 for the possible future town. The per capita domestic demand was estimated at 50 gallons per day; industrial requirements were estimated and provided for in event of some future developments.

A spring with a minimum capacity of 250 gallons per minute located on Clear Creek was chosen as the source of water supply. This water is treated at the pump house by chlorination. The capacity of the chlorination equipment ranges between a minimum of 10,000 gallons per day and a maximum of 1,650,000 gallons per day. Three centrifugal pumps, having a capacity of 567,000 gallons per day, were provided for pumping. The demand in July 1935 was about 275,000 gallons per day.

A covered concrete reservoir 60 feet in diameter and 15 feet high with a capacity of 250,000 gallons is used for storage. The main supply pipe, from the spring to the pump house, the pump house to the reservoir, and the reservoir to the town distribution system is all 8-inch, cast-iron, cement-lined pipe. Pressures in the distribution system range from 300-foot head to 400-foot head, depending on location. Copper tubing is used for service connections.

For fire protection, three 250-gallon-per-minute streams with a minimum hydrant pressure of 20 pounds per square inch were provided. There are 46 hydrants spaced throughout the village and camp at intervals of from 500 feet to 600 feet apart. The hydrants are of the post type with 2½-inch outlets. A truck-pumper with a 650-gallon-per-minute capacity and 1,000 feet of 2½-inch hose and 250 feet of 1-inch hose is provided for fire fighting.

Sewage-disposal system.

In the design of the treatment plant, a maximum daily flow of 200,000 gallons was assumed, based on a population of 2,000 at 100 gallons per capita. Provision was made for possible future expansion to take care of a population of 2,500. Infiltration of ground water into the collection system was assumed to be negligible.

The treatment plant is designed to give the fullest practicable degree of treatment by biological process. This high degree of treatments is required because of the low flow in the stream which receives the plant effluent. The disposal process used at this plant includes primary sedimentation, activation, and aeration by means of recirculation, secondary sedimentation with sludge concentration by

means of gravitation, spray aeration, and sludge-drying beds of the open type.

The sewage enters the plant in a header designed to distribute the flow evenly and with low velocity into a battery of six Imhoff tanks, each being a separate and complete unit.

The collection system was designed to handle only the sanitary sewage. Minimum sizes specified for the various types of collection pipes are: service connections, 4 inches; service laterals, two or more houses, 6 inches; mains, 8 inches. The limiting grade (minimum) for laterals is 2 percent; collecting mains minimum, 0.5 percent; outfall line minimum, 0.3 percent.

After the construction of the dam was completed and the load on the plant decreased, the treatment method was revised. At present the treatment consists of primary sedimentation, activation, aeration, secondary sedimentation, and sludge-drying beds. A high efficiency is obtained by this method of operation.

Electrical distribution system.

The design of the electrical distribution system was complicated by the delay in making a decision as to the source of power, indecision as to the type and number of housing units to be built, and the decision to use electric space heating in many of the structures after the preliminary design and construction of the system had been started. Accordingly the cost of the completed system reflects the influence of these various factors.

Prior to the construction of the town or the dam, a local power company had built a substation just north of Norris Dam, on the east side of the Clinch River. Unfortunately, this substation was placed in such a location that it would be flooded after the completion of the dam.

A 2.3-kilovolt auxiliary line extended from this substation past the town site to State Highway No. 61, then along this highway to the communities of Glen Alpine and Andersonville, serving rural customers in the area traversed. That part of the 2.3-kilovolt line extending from the substation at the dam to the town site was purchased by the Authority and the first power for the town was carried by this line. Later the Authority constructed a substation on the west bank of the Clinch River above the dam, converted the 2.3-kilovolt line to an 11-kilovolt line, and dismantled the old substation above the dam. In the meantime, construction operations at the dam and at the town, the trade shop, the construction camp itself and the temporary water pumping plant were served by temporary 2.3-kilovolt lines that were extensions of the 2.3-kilovolt auxiliary line.

When the 2.3-kilovolt line was converted to 11 kilovolts, two substations were constructed at the town itself, transforming voltage from 11 kilovolts to 2.3 kilovolts, and the distribution system extending along State Highway No. 61 from Andersonville to Glen Alpine continued to operate on 2.3 kilovolts from one of the town substations.

After the primary conversion of 11 kilovolts the load side distribution in Norris was converted from a 2.3-kilovolt 3-wire to a 4-kilovolt 4-wire neutral ground system. This type of system seemed necessary on account of the tremendous load resulting from electric space heating.

After the dam was completed and put in operation the town electric system was served directly from the dam by an 11.5-kilovolt line. This circuit extended from the dam to the two primary substations within the town, where the voltage was transformed to 4 kilovolts for distribution. The major part of the town was then served by a 3-phase, 4-kilovolt line. The current for customer use was reduced by three 37.5-kilovolt-ampere transformers connected in wye on the primary and secondary sides and located on separate pole structures. All three phases and the neutral, which was common to both primary and secondary, were brought into the customer's entrance. The distribution transformers were connected to give 120 volts to ground and 208 volts phase to phase. Three phases were brought to the customer to reduce conductor size and to equalize the load between phases.

One section of the town was served by single-phase 2.3-kilovolt primaries and standard 115/230-volt distribution transformers.

Town substation No. 1 contained two banks of three 200-kilovolt-ampere transformers and substation No. 2 contained one bank of three 200-kilovolt-ampere transformers. The three banks operate independently, but any two, or all three, may be paralleled if so desired.

The conductor of the 11.5-kilovolt feeder from the dam to the town substations was number 1/0 copper. The 4-kilovolt distribution system was number 2/0 copper, with the exception of the primary feeding the single-phase part of the system, which was number 4 copper. The 208-volt secondaries were number 4/0 copper and the 220-volt secondaries in the single phase system were number 2/0 and number 2 copper.

At the completion of the system there were 63 transformers with capacities of 2,100 kilovolt-amperes serving the residential load in the town. There were 34 transformers with a capacity of 1,815 kilovolt-amperes serving the commercial and industrial load of the town.

After virtual completion of the town system, the section of line serving Andersonville and Glen Alpine was purchased by the Authority. In addition to this extension to the town system two short extensions were constructed, which consisted of 3 miles of single-phase rural line, and were served from the town's 4-kilovolt distribution system.

At the present time the town system is fed directly from the dam with an 11.5-kilovolt circuit. Rural lines which have been constructed in this area in the interim are served from a separate 11.5-kilovolt circuit, which is parallel with the town circuit. In the event of line trouble on either the town circuit or the rural circuit, the town or rural load may be picked up by the other circuit, through switching arrangements at the town site.

CONSTRUCTION CAMP

General lay-out.

The construction camp was located at the west edge of the town, primarily because this location gave the most convenient access to the dam. The general arrangement of buildings was established by selecting the tops of two ridges which came together to form a **V**. At the apex of the **V** the road was laid out to form a rectangle en-

closing the top of a knoll and two streets came into two corners of the rectangle from the two ridges. Originally these two streets were designed of nearly equal lengths, but, owing to a change in the estimated requirements, the southern road was stopped short and terminated with a small turn-around.

In the most commanding position at the apex of the V, a community building was located overlooking the square and opposite the cafeteria building. At the far end of the turn-around was a women's dormitory, with a staff dormitory on the south side of the short street. There were six workmen's dormitories of similar design, four of which were placed in two pairs on either side of the long street, and the other two on the otherwise unoccupied sides of the rectangle. This left space for at least one additional dormitory on the long street if it should have proved necessary. The total capacity of the construction camp as built was approximately 925 persons.

Design.

It was difficult to forecast the requirements that the construction camp would have to meet as the employment load and construction schedule for the dam had not been fully formulated when design was begun. Before determining the capacity of buildings it was necessary to consider the amount of shelter available in the region, the number of shifts of labor, and the number of hours to be worked per day per shift. The number of workmen's dormitories originally planned was eight. This was reduced to six in order to avoid risk of surplus. The six workmen's dormitories built, sheltering 118 men each, proved to be inadequate during a short period at the peak of employment, and for this reason basement space was developed wherever practicable. At the peak of employment it also became necessary to erect about a dozen portable huts in the rear of the dormitories and also to build a group of cabins which could later be used for tourists.

It would have been necessary to have accurate knowledge about the shifts of labor and hours of departure to and return from the dam to estimate accurately the cafeteria capacity required. This information was not available at the time the camp was being designed, and the final result indicated some over-capacity in the dining space and plant as installed.

The lay-out of the camp proved to be convenient, and the buildings were so related to each other as to have an orderly and cumulative effect. A minimum distance of about 75 feet was maintained between buildings for fire protection. The amount of set-back from the street was carefully correlated to get the most desirable result. The natural grades were preserved for the most part, and in this way it was possible to save the trees which contributed a great deal to the atmosphere of the camp.

The community building was first conceived as a meeting place and recreation center intended mostly for men of the camp. At this time a separate community center was to be built to serve the village. As the design progressed, decisions relative to a separate village community center were postponed so that the community building in the camp had to serve as a community center for both camp and village for a considerable period of time. From the summer of 1934

to the summer of 1935 the facilities were taxed to the utmost, several successive meetings sometimes being scheduled for each room in one evening.

The type of construction in the camp was predicated on temporary use of the buildings; however, on account of numerous reports of termite difficulties in the region, the buildings were placed on brick foundations, and while the construction was rough, especially as to interior finish, it was sound construction and proved to have a longer life than the actual need of the job required. If a continuing use had been definitely foreseen, these large wooden buildings would have been designed with a better standard of safety from fire hazards. The original design for dormitories provided for one-story buildings of H plan. Owing to the restricted area and lack of flat ground that could be developed, a long, narrow two-story dormitory was adopted instead, although it undoubtedly involved some fire risk. All ordinary safety precautions were taken, and in addition the building was further guarded against possible fire outbreaks by frequent inspection and patrolling at night.

The question of heating received considerable attention, but time did not permit a very deliberate study of the problem as the materials had to be scheduled some time in advance of construction. In actual practice the individual hot-air heating plant and forced-draft system installed delivered enough heat, but not in a very hygienic way. In the dormitories the warm air was distributed through ducts discharging into each room near the ceiling and the cold air returned under the doors of the cubicles which were left 6 inches off the floor for this purpose. From the corridors the return air was exhausted by ducts to the basement. Electric heating of the construction camp buildings had not yet been investigated enough to be used in this camp.

Considerable attention was given to external color treatment of the buildings, which was accomplished by using a creosote stain.

The design of the construction camp began in September 1933. Construction of the workmen's dormitories began during October and they were occupied about 90 days later. The entire camp was substantially completed about May 1934.

Workmen's dormitories.

Each of the six workmen's dormitories was a 2-story building. Habitable basements were developed where irregularities of grade made this possible. In the first and second story of each dormitory there was a longitudinal corridor with stairways at the middle and at the two ends. The capacity of the first and second stories was 118 men, housed in 2-men cubicles, approximately 9 by 8 feet each. The basements were not divided into cubicles, but were simply open spaces in which bunks were arranged in aisles. A large washroom was provided in the center of the building on each floor. A table was built in each of the cubicles. Windows were check-rail wooden sash, 1 per room.

Construction above the foundation consisted of light wood framing sheathed with vertical boards and battens in the first story and

faced with oak shingles in the second story. Interior partitions were lined with shiplap on one side only. Floors were common oak, and the roofing was asphalt shingles.

After these dormitories had outlived their usefulness, at the completion of the construction program, four were demolished by Civilian Conservation Corps labor. All plumbing material, electrical equipment, and other salvageable material that could be used by the

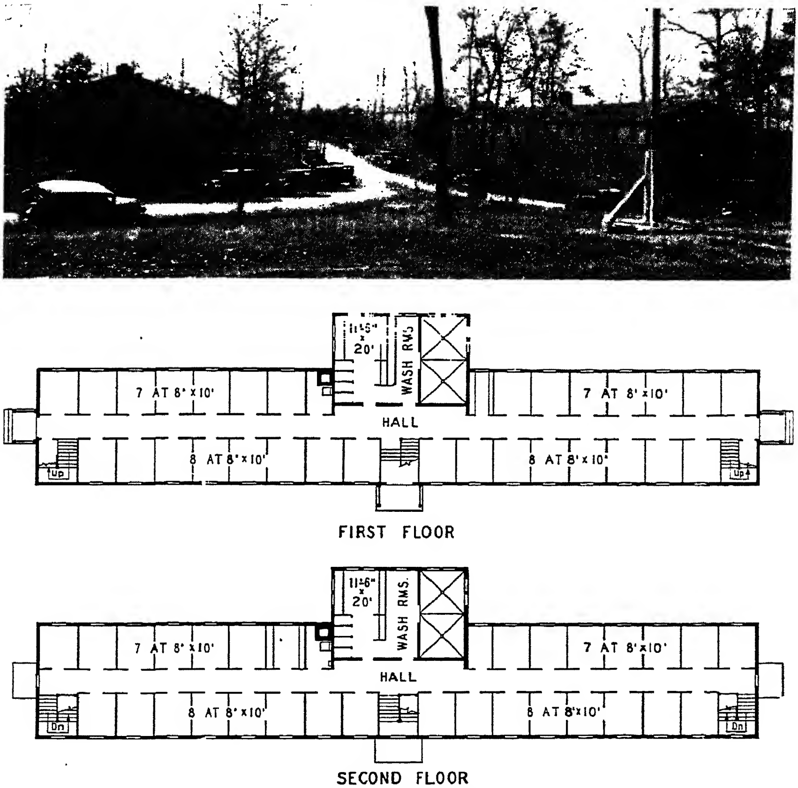
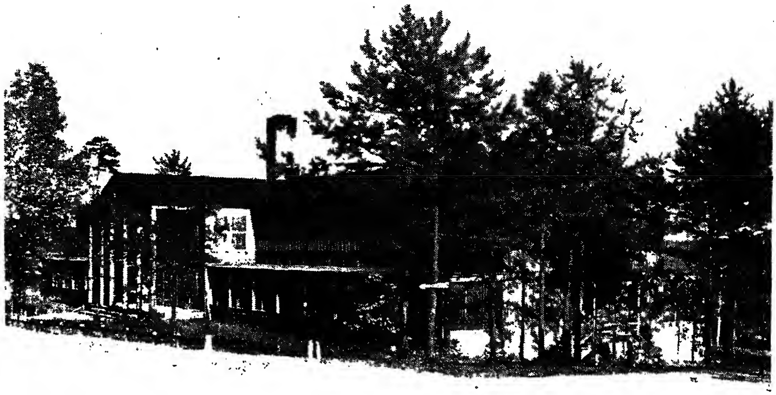


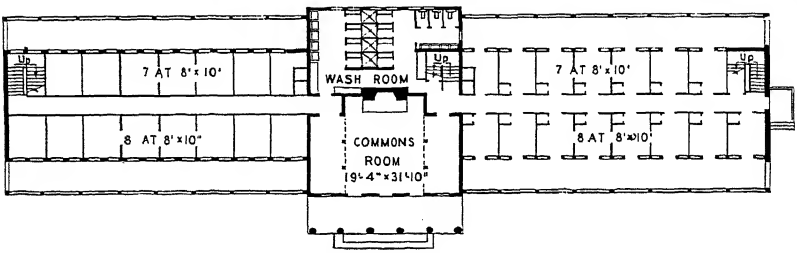
FIGURE 76.—*Typical workmen's dormitory.*

TVA was removed and delivered to the warehouse for possible use at Gilbertsville and other projects. Lumber and other materials were transferred to the United States National Park Service for use in building Cove Lake State Park at Caryville and Harrison Bay State Park near Chattanooga. These parks are operated by the State of Tennessee. The C. C. C. forces landscaped the site where these dormitories stood, putting it back in its natural state as far as possible.

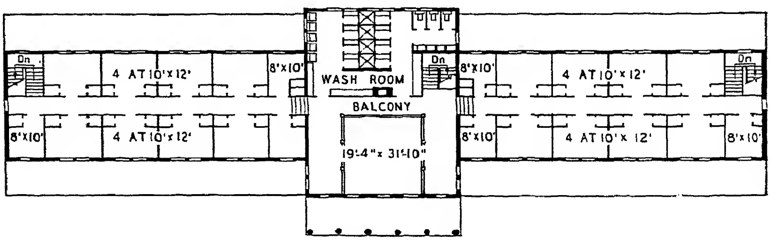
Staff dormitory.

This building was designed to house engineers and others who required more living space than the workmen's dormitories provided. This extra space was particularly useful in providing space for mem-





FIRST FLOOR



SECOND FLOOR

FIGURE 77.—*Staff dormitory.*

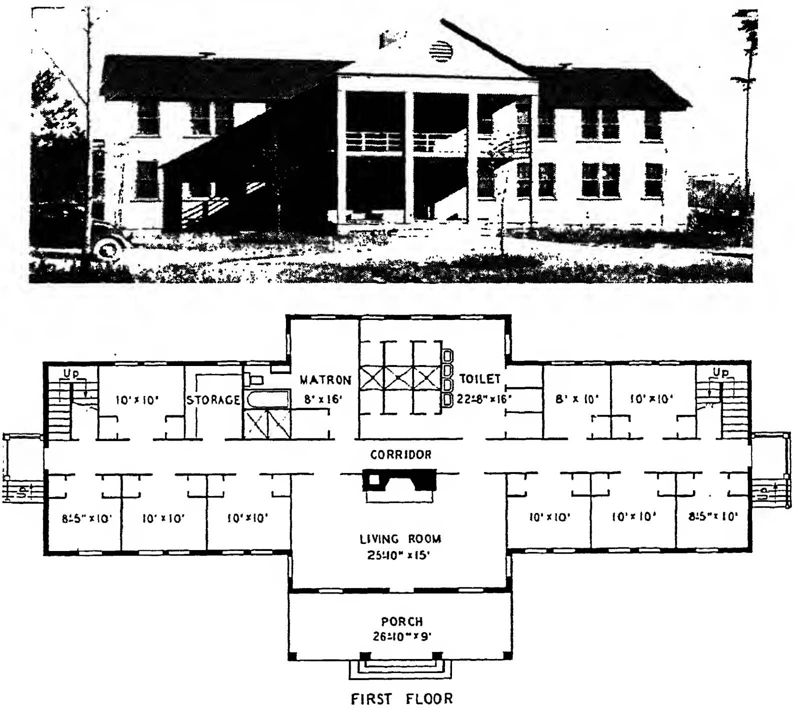
bers of the staff to work after hours. There were 42 sleeping rooms of varying sizes, designed partly for single and partly for 2-men occupancy. A centrally located lounge extended the full height of

both stories and a gallery around 3 sides at the second-story level was provided. Along the entire front and back of the building sleeping porches intended to serve the rooms of the first story were provided. Part of this building was occupied as an infirmary during the first part of dam construction as no hospital was built.

The building had a brick foundation and was of light wood frame construction similar to the workmen's dormitories. The entire building burned to the ground after its use as a staff dormitory had terminated.

Women's dormitory.

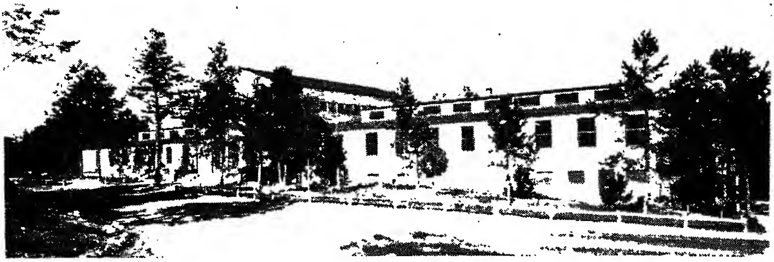
The women's dormitory was similar in construction to both the workmen's dormitories and the staff dormitory. The building was remodeled for use as an infirmary during the latter part of the dam

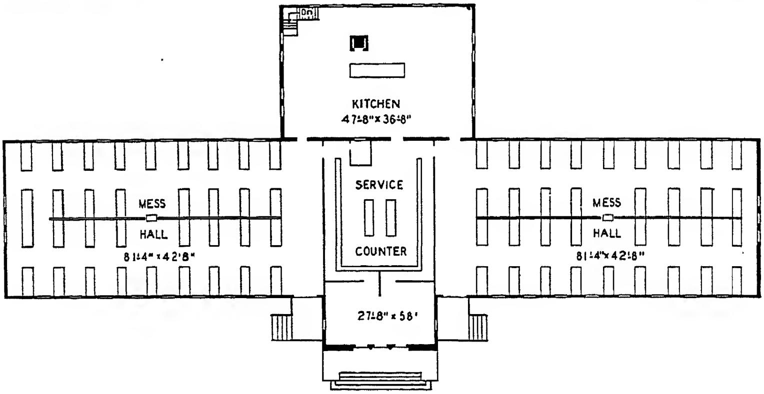


NOTE SECOND FLOOR SIMILAR IN PLAN TO FIRST FLOOR

FIGURE 78.—*Women's dormitory.*

construction. At that time the women employees were housed in one wing of the staff dormitory which had been previously assigned to the engineers. The two wings were separated by a partition at the west end of the lounge.





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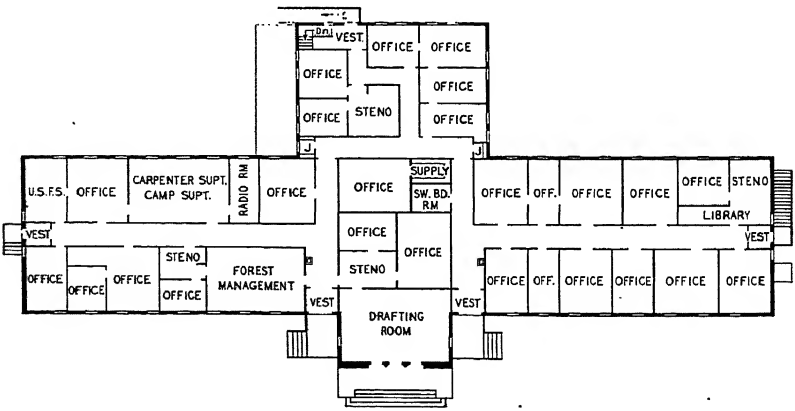

B

Figure 79.—*Cafeteria building as used during dam construction and after adapting for office building after construction.*

Cafeteria.

Because four principal shifts and a number of smaller shifts would be coming and going at different hours of the day and night, and meals would have to be served at irregular hours, a decision was made to lay out this building for cafeteria style of service. The right and left wings were dining halls with a double serving counter in the center, and a kitchen extended to the rear of the building, forming a T. A clerestory was provided over the dining wings and over the kitchen so as to provide extra light and ventilation. The lay-out of the tables was related to the rows of group columns in such a way as to avoid interference. In each dining wing there were four rows of tables, two rows being adjacent to the front and rear walls, the other two being adjacent to each other but separated by longitudinal dividing partitions. A water station and coffee urn were provided in the center of each dining wing.

The entrance to the building projected from the center so as to give space to shelter men while waiting in the serving line during inclement weather. The serving counter was U-shaped with the cashier at the center of the closed end. A double service line was used. The serving counter was provided with a steam table and other necessary equipment and the kitchen was equipped for electrical cooking. Food was prepared in a room in the basement under the kitchen which was reached by a stairway and served by an electric dumb-waiter. The seating capacity of this cafeteria was 544 persons; the maximum number served at any one meal was 900. During the peak month approximately 90,000 meals were served.

The building was of light wood frame construction on brick foundation. A linoleum covering was laid on the floor of the kitchen and serving space. The flooring in the dining wings was common oak.

The cafeteria building was remodeled after construction of the dam had been completed and made into offices, as shown in figure 79.

Community building.

This building, as the name implies, was designed to serve a number of purposes. In the center of the front was an entrance lobby (later occupied in part by a temporary post office). At the right as one entered was a commissary and lounging space, and at the left was a library with two reading rooms. The main body of the building in the rear of the lobby consisted of a combination gymnasium and auditorium with a seating capacity of approximately 600 persons. On one side of the combination gymnasium and auditorium across a corridor was a small auditorium seating approximately 150 persons, and in the corresponding position on the other side were washrooms and small office rooms. Other parts of the building were later partitioned off to form additional office rooms.

The construction of the building was similar to the rest of the construction camp except that the roof of the auditorium was supported on steel trusses and steel columns.

This building was later partially remodeled after the construction at the dam was completed and made into a town restaurant, and space was provided for a Federal art project and for concessions.

town. The majority of houses were set with the longer dimension parallel to the road. This generally gives dignity but is possible only where there is sufficient space for such development. A large number of houses, however, were set with the gable end toward the street, and these were interspersed with the others in such a way as to give interesting contrast. A few houses were set in staggered relation to each other and not parallel to the street.

A notable characteristic of the town which contributes to its special atmosphere is that no property lines have been indicated between the house sites. It was at first thought that a division into individual house lots would be required, but in the several years of occupancy no such demand has ever developed. In consequence, a pleasing parklike open atmosphere has been obtained which would not have been possible with individual properties separated by fences and hedges. Improvements around each house contribute to, rather than compete with, neighboring improvement. The houses are so related to each other that the service side of one house does not detract from the main living side of neighboring houses, and in fact the houses for the most part are presentable in appearance from all directions. While the soil did not lend itself readily to cultivation, the lack of luxuriant cultivation has been overcome by careful preservation of the surroundings as they were found in their natural state.

The 5 apartment buildings were grouped together on a high ridge away from the single-family houses. The 10 duplex houses were built more or less in groups and interspersed throughout the town. They appear somewhat like a single-family house and their presence does not detract from the appearance of the town as would the multifamily buildings.

Architecture.

Group 1.—After a rapid but comprehensive canvass of the various alternatives in architectural design, it was decided that every house planned should have a strictly rational layout as its keynote, and that the exterior development of the house plan in three dimensions should be a straightforward expression of the plan in a simple and homelike form. Reduced to bare essentials, this meant generally a rectangular plan and a simple pitch roof without any important featuring of dormers. Both the plans and the elevations were studied carefully in relation to each other to produce a satisfactory effect by simple mass and proportion. No superfluous architectural elements were added for embellishment. The fenestration in many of the houses has a modern feeling in that windows are used which are wider than they are high in order to provide ample ventilation in the summer. However, the extreme modern peculiarities of recent types of fenestration were not considered suitable to the design.

The houses were built without cellars; and as most of them had wood-floor construction, and because of possible termite infestation, it was considered essential to provide ample ventilation under the floors. To do this a minimum clearance of 1 foot was provided between the under side of the joists and the high point of grade

under the house. It was not deemed advisable to scoop out the natural grade to any great extent under the house, because the foundation wall would then have to be strong enough to serve as a retaining wall, and because such a depression would be apt to accumulate a pool of water under the house. Where the grade at the house site sloped considerably, the floor would be too far above the low point for convenience and pleasing appearance. To meet this situation two types of hillside houses were designed and used which had staggered floor levels more or less matching the slope of natural grade. These floor levels were developed for convenient living arrangement in the house.

Entrances to all the houses were studied in relation to site conditions and many special adaptations were designed and used. A liberal use was made of porches in all except a group of 80 cinder-block houses which were governed by rigid considerations of economy.

The houses varied in size from three to eight rooms, with four rooms and supplementary space in the attic representing about the average condition. The sloping roof, one and one-half story height, and habitable attic space—features adopted for a large portion of the houses—provided a homelike appearance. The use of insulation in walls, roof, and ceiling made possible a habitable and economical development of the attic space which was quite generally used as unpartitioned sleeping space. During the time the houses have been occupied, however, most of these attics have been finished to give two or three additional rooms.

Several principles were uniformly applied in developing the ground floor of the houses:

1. The rooms were to be of sufficient size but not expensive.
2. The rooms were to be conveniently related to each other, with access from each room to the bathroom without passing through another room.
3. Ample closet and storage space with no lost space was to be provided, and the entire plan was to be enclosed in a shell of simple architectural form.
4. The dining space was to be combined economically with either the kitchen or the living room, rather than occupy a separate room.
5. On account of the warm climate in summer, rooms should have cross ventilation or corner ventilation wherever practicable.

Before it was decided to install electricity for heating, cooking, hot water, and refrigeration, the plans for the various house types were developed with a thought of using fireplaces and stoves for heating and cooking. This controlled the relationship of the various elements designed. In particular it affected the arrangement of the kitchen as the range had to be next to the flue, and the kitchen flue in general was in the same chimney with the living-room fireplace flue. When these considerations were removed, many of the relationships in the preliminary plan were no longer logical, and a general revision was necessary. However, some traces of the original commitment remained in the designs as executed.

In developing a particular form of electrical heating, and in arranging a plain and simple type of housing for the use of modern electrical equipment, a considerable amount of special study was

necessary. Other technical features that had unusual importance were the use of up-to-date plumbing (including development of a novel type of aluminum shower bath stall), modern lighting, wall insulation of an advanced type, and termite protection by means of projecting copper shields from the top of the foundation walls. Although some of these features were experimental at the time, they proved generally successful. A new type of interior finish, using a shiplap wainscot 3 feet high, jointless plywood walls above, and insulation board ceiling with regularly spaced joints, was employed. This early use of plywood interior finish was a forerunner of rather extensive later use in other places.

The architectural treatment of these houses did not call for any considerable amount of interior paint. Ceilings were left unpainted, the wood trim was given a natural finish, and the plywood was finished with a light coat of creosote stain. A great deal of attention was given to the exterior color treatment, with variety attained by leaving many houses in the natural color of the materials and by painting or whitewashing others. Special groupings of houses were emphasized by homogeneous color treatment.

Quantitative data for this first group of 152 houses is given in table 38.

TABLE 38.—*Quantitative data—first group of 152 houses*

Type	Number of houses built	Number of stories	Number of rooms [1]	Cubic feet	Square feet of floors	Ratio of cubic feet to square feet
D1, D1M	11	1½	5	14,650	990	14.8
D2, D2M	15	1½	5	14,650	1,010	14.5
D2S, D2G	9	2	6	15,650	1,310	11.9
N2, N2A	9	1	4	13,000	750	17.3
21	10	1	3½	10,300	684	15.0
22	1	1	2½	9,500	638	14.9
32	13	1½	4½	13,700	783	17.5
33	6	1½	5½	15,700	1,160	13.5
34	4	1	3½	12,600	829	15.2
41A	3	1½	6	17,500	1,317	13.3
41B, 41E	7	1½	6	17,500	1,292	13.5
41C, 41D	20	1½	6	17,900	1,307	13.7
41F	1	2	8	18,700	1,872	10.0
42	8	1	4½	16,000	1,011	15.8
43	8	1½	5½	15,400	1,126	13.7
44, 44A	11	2	5½	14,300	1,345	10.6
45	1	1½	6	24,000	1,681	14.3
46	1	1½	4	14,000	1,100	12.7
54	9	1½	6½	18,700	1,418	13.2
55	1	2	8	21,700	1,640	13.2
56	1	1½	6	16,500	1,205	13.6
57	1	1½	5½	15,400	1,260	12.2
63	1	1½	7	18,800	1,476	12.7
Steel	1	1	3	5,750	500	11.5
Total	152		772	2,320,000	166,328	

[1] Or equivalent quarters.

Typical house types are shown in figures 81 to 87.

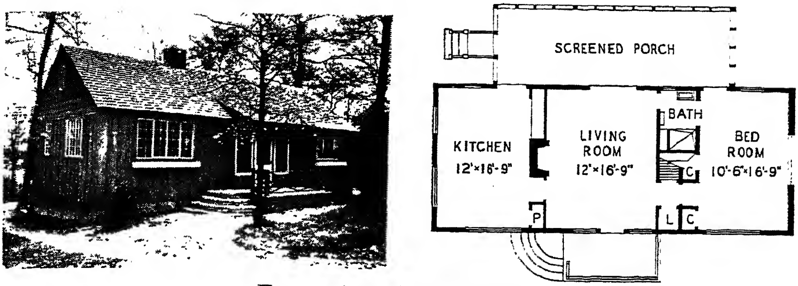

FIGURE 81.—*House type D2.*

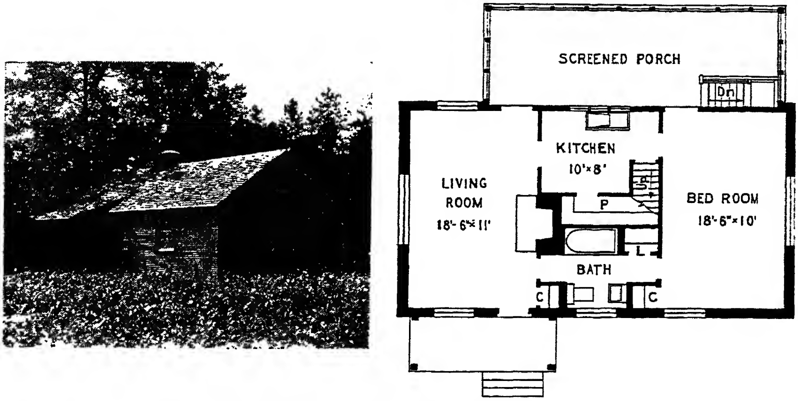

FIGURE 82.—*House type N2.*

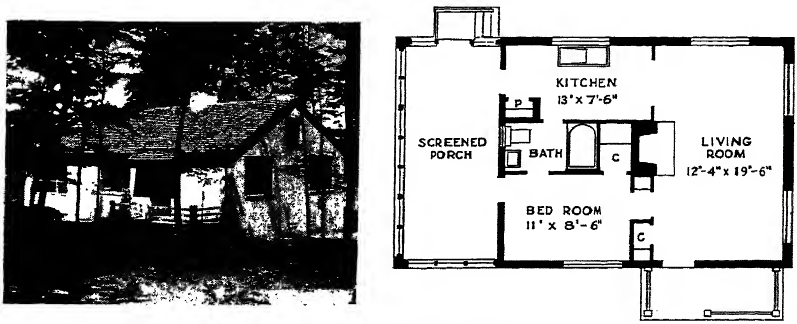

FIGURE 83.—*House type 21.*

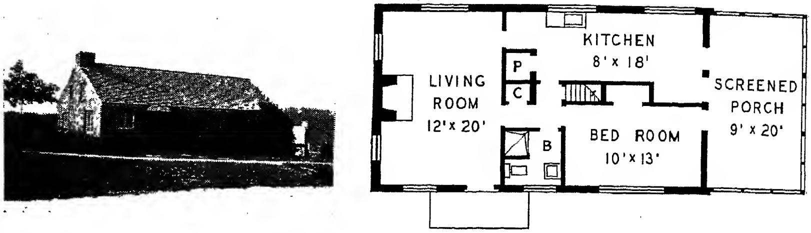

FIGURE 84.—*House type 32.*

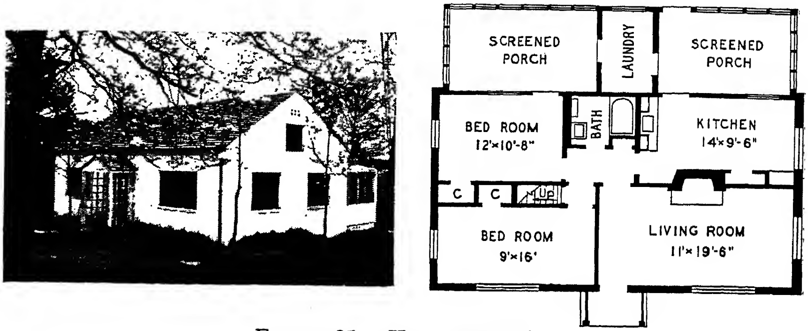

FIGURE 85.—*House type 41C.*

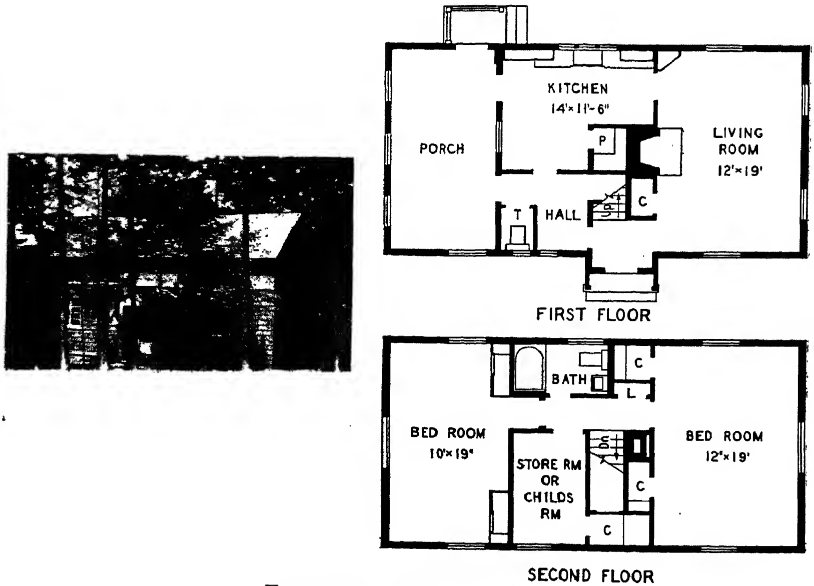

FIGURE 86.—*House type 44.*

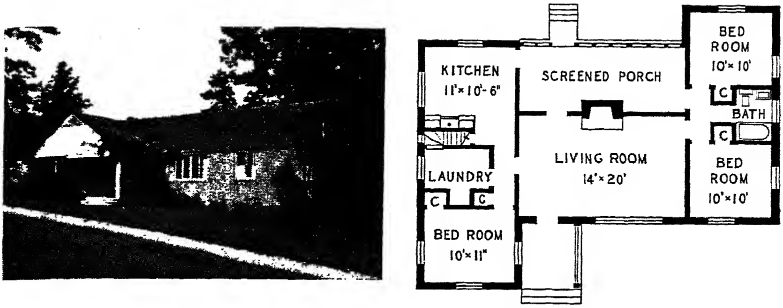

FIGURE 87.—*House type 54.*

Group 2.—The 130 houses constructed in this group were started about 6 months after group 1. Eighty (types A, B, C, and D) were finished and ready for occupancy in October 1934. The remaining 50 were designed in the summer of 1934 and built during the following winter. The first 80 houses were located on Garden and Oak Roads, two winding intersecting streets on hilly ground. The others were interspersed through the part of town previously developed. Quantitative data for these houses is given in table 39.

TABLE 39.—*Quantitative data for the group 2 houses*

Type	Number of houses built	Number of stories	Number of rooms	Cubic feet	Square feet of floor	Ratio of cubic feet to square feet
A	30	1	4	8,424	624	13.5
B	10	1	4	8,470	605	14.0
C	20	2	4	10,500	1,000	10.5
D	20	2	5	10,375	1,000	10.4
K-C	49	1	4	9,600	662	14.5
Guest house	1	1	2			

These houses all employ a distinctive method of construction developed by the Authority. Types C and D were just under two full stories in height and have large rooms while the others are one story in height and have smaller rooms. Attic space is for incidental storage only, with access by trap door. None of the houses has a basement, and porches were eliminated except for type K–C. They are arranged for heating by coal or wood stoves.

A precast beam and slab floor was developed and used in construction rather than a field-poured slab. Since the concrete floor and wall unit comprise almost the whole structure up as far as the attic, this type of construction may be considered fire-, termite-, and rodent-proof. The structural surfaces are the finished surfaces, and no superficial finish was applied although provision was made for an exterior coat of stucco if later required. Experience indicates that the floors and walls are satisfactory as built for the purpose intended, and they warrant a reasonable expectation of permanence and weatherproof qualities. The walls have enough thermal insulating value so that these houses are comparatively easy to heat.

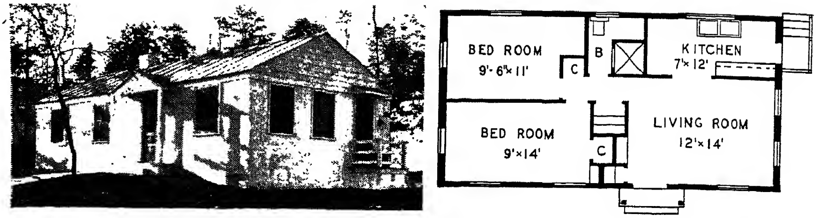

FIGURE 88.—*House type A.*

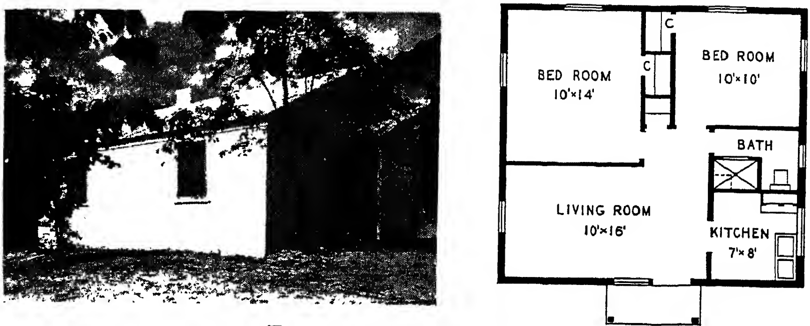

FIGURE 89.—*House type B.*

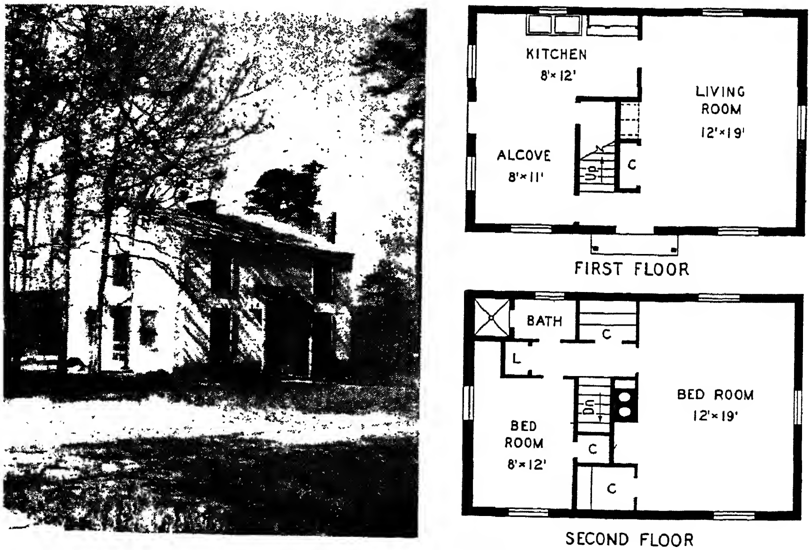

FIGURE 90.—*House type C*

The wall blocks required some experimentation to obtain an aggregate and texture that would be moistureproof, that would take and hold nails, and that would have a satisfactory appearance when finished with one heavy coat of cement paint. For maximum economy in laying, the especially large size blocks used in types A, B, C, and D were abandoned in favor of standard size blocks in type K–C. These houses were equipped with plumbing, electric lighting, and a coal range for heating and cooking. Types A, B, C, and D also have flues for heating rooms by stoves while the K–C houses have a fireplace in the living room. A hot-water tank is placed to provide heat in the bathroom.

These five types, A, B, C, D, and K–C, are shown in figures 88 to 92.

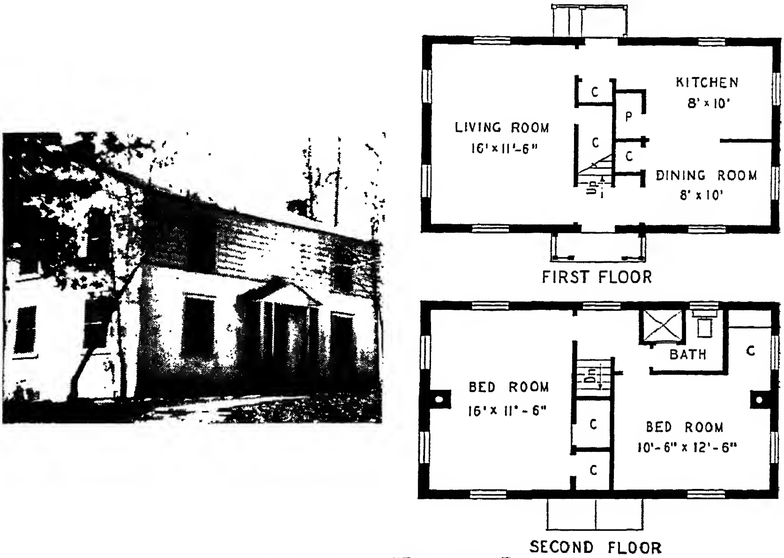

Figure 91.—*House type D.*

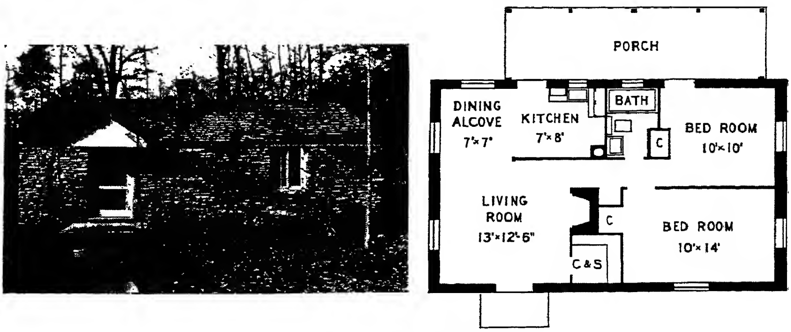

Figure 92.—*House type K–C.*

Duplex.—The 10 duplex houses comprising 20 units were all 1-story frame construction; each unit having a living room, a bedroom, a kitchenette, and a bathroom. They were of low-cost construction and were arranged for heating and cooking by coal or wood stoves.

Apartments.—Five apartment buildings with 30 one- to four-room units were built. There were 3 buildings with 4 units each, 1 with 8 units, and 1 with 10 units. The buildings were of frame low-cost construction. The interior finish was similar to the finish in the group 1 houses. Electrical cooking facilities were provided and the apartments were heated by a central steam-heating plant.

Remodeled farm houses.—Some of the better farm houses in the town-site area were remodeled and used. In most cases modernization was the only change necessary, and in all cases the houses fit extremely well into the town plan.

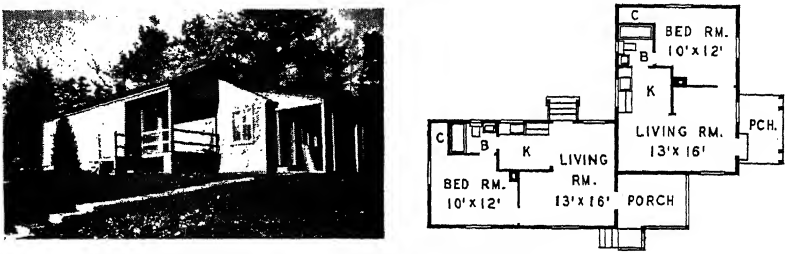

FIGURE 93.—*Typical duplex house.*

Construction.

Brick veneer construction was adopted for a large portion of the group 1 houses on account of its permanence. Some houses with wood exterior finish were used for economy and variety, and the informal board and batten treatment of local cabins was used effectively in some designs. A few stone-veneer houses were built, but the number was limited because the stone of the locality was not economical to work. One experimental house of sheet steel was purchased to test its inherent economy and qualities in actual service. The walls and other structural parts were purchased and erected by the manufacturer, but the Authority did all of the other construction work.

In the group 2 houses the successful use of cinder-concrete block walls in a large number of houses constitutes an important innovation. This is significant in that a permanent structural material was used in which the structural surfaces also served as the finished surfaces inside and outside the house. Special precautions were necessary to make such a wall reasonably weather resistant. This bare type of building was adequate for minimum housing. Provision was made, however, for subsequent improvement for a higher rental occupancy later. The improvement has already been made in the two-story cinder block houses by insulating and lining the inside of the walls and installing electrical equipment. The original construction anticipated improvement outside the house if it should ever be required by applying stucco to the cinder blocks, and provision for this was made in the structural details. The special type of floor

employing precast concrete beams and integrally-colored precast and highly-finished cement floor slabs is also a striking innovation designed for low maintenance cost.

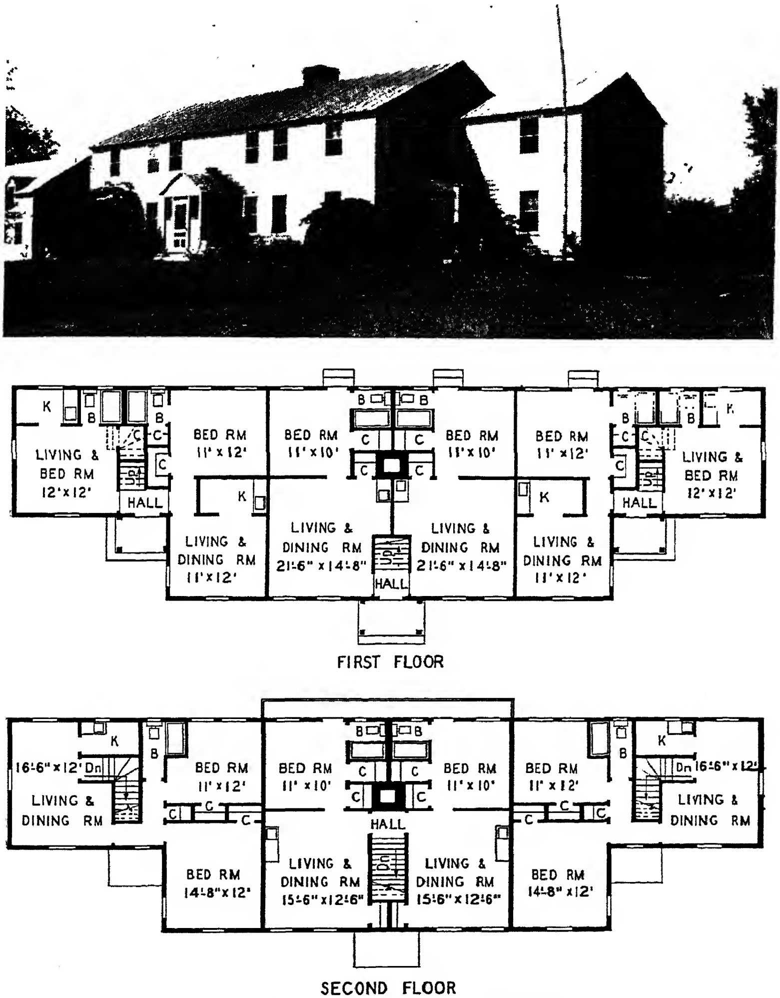

FIGURE 94.—*Typical apartment building.*

The remaining residential buildings—duplex houses and apartment houses—were of low-cost frame construction similar to the group 2 frame houses.

The construction work in the town was done by the Authority on a force account basis, with few exceptions.

Landscaping.

Grading around houses was kept to a minimum because of the cost and because of the undesirability of exposing large areas of bad subsoil. Grading was also kept low in order to prevent the creation of extensively steep banks on topography which to begin with usually had considerable slope, and to avoid excessive cutting of trees around house sites and wooded areas. Generally, top soil was not worth stripping and saving, but it was preserved whenever feasible. As far as possible, spreading heavy subsoil on the surface was avoided. In a few instances, inexpensive dry-rock retaining walls were used where relatively heavy cuts were unavoidable at the rear of houses.

FIGURE 95.—*Typical remodeled farm house.*

To improve the soil, in large open areas cowpeas and Johnson grass were sown for summer growth and then plowed under in the fall. Thereafter, domestic ryegrass was sown, together with permanent grass mixtures, including lespedeza for spring planting. In smaller open areas around individual houses some top soil was provided and seeding done. Additional improvements were left to the tenants. For access from sidewalk or street to house entrances, stepping stones were provided. Service paths to kitchen entrances were surfaced with crushed stone chips.

Planting around individual houses was left to their occupants who were given the opportunity of selecting adequate quantities from a supply of trees and shrubs purchased for that purpose. Advice about planting was furnished by a resident landscape architect where such advice was requested. Plant materials have continued to be available to those who wish to improve the properties occupied to an extent greater than the minimum needed for foundation planting of the house and its immediate vicinity. Planned planting was carried out on the grounds of the school, the community center, the five apartments, the town common, and in large open areas throughout the town.

Garages, service roads, and parking space.

The frequent steepness of slopes between houses and streets, combined with desirability of keeping costs as low as possible, created a problem regarding the location of garages and entrance drives to dwellings set some distance back from the street. To service the first group of 152 houses, group garages were located in the block interiors and reached by one or more drives from adjacent streets.

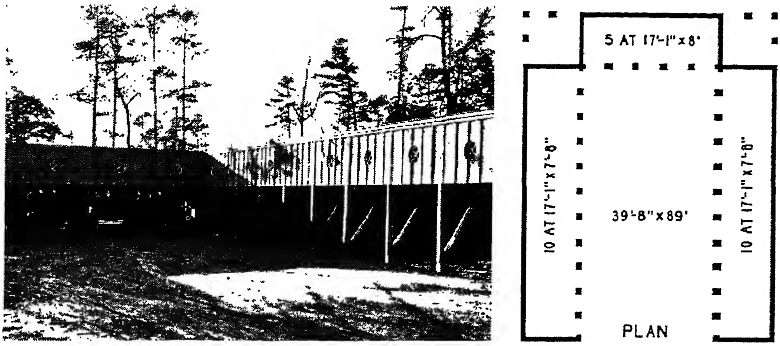

FIGURE 96.—*Typical group garage.*

Later, when some of the houses were designed to be heated by coal rather than electricity, the need of providing access for coal trucks to reach each house caused the building of service roads. These were located unobtrusively, providing a turn-in to each coal-burning house. Outside storage of coal was taken care of by means of bins consisting of three precast concrete slabs for sides. Group garages for these utilized existing service drives and were located as inconspicuously and as close to the houses served as possible.

In comparing the effectiveness of large group garages with that of smaller units, there is evident need for balancing the relative inconvenience of greater distances having to be traversed by some car owners in reaching isolated group garages in the center of blocks with the undesirability of having the smaller multiple unit garages rather close to some of the houses. Neither solution is ideal, but group garages cost considerably less than providing an individual garage and entrance drive for each house. In many cases it was practicable to provide parking areas adjacent to main roads and service drives, where many occupants prefer to leave their cars at all hours rather than avail themselves of garage space.

Athletic and recreation facilities.

The community building and play areas, which included softball and baseball diamonds, tennis and volley ball courts, and children's playgrounds, offered facilities for wholesome recreation activities. This program by the Authority, sponsored in cooperation with a

voluntary employees' recreation association, included outdoor and indoor activities of all kinds, including music, athletics, dramatics, hiking, fishing, and boating.

Parks and playgrounds.

The town development included an adequate yard with each house as well as frequent undeveloped woods or open areas close at hand, and for this reason it seemed unnecessary to develop any parks as such within the town. Norris Park is only a short distance from the developed portion of the town to the north and west. On the other side of the town the protective belt offers unlimited opportunities for walking and children's play. Adjacent to the school a large playground was provided, including separate play areas for kindergarten and nursery school children, for the smaller grade school children, and for those of high-school age. Another smaller playground in the center of the group of cinder-block houses on the south side of the town includes a wading pool, swings, and other types of apparatus.

Costs.

The total costs for all buildings and groups of buildings comprising the residential section are shown in table 40. These costs are further analyzed on page 218.

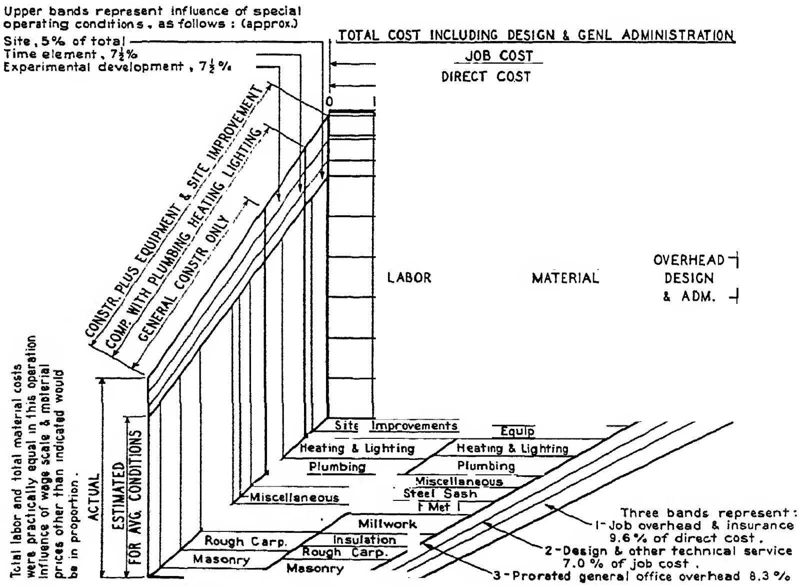

FIGURE 97.—*Cost structure of group 1 houses.*

The weight of the various factors making up the cost of the two large groups of houses is shown diagrammatically in figures 97 and 98.

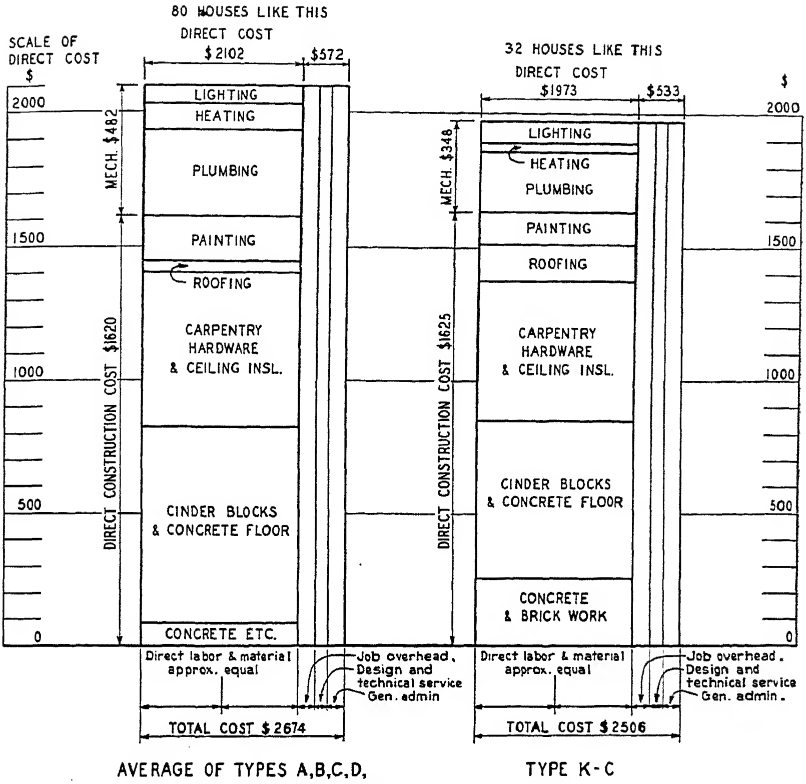

FIGURE 98.—*Cost structure of group 2 houses.*

TABLE 40.—*Cost comparisons of houses in the residential section*

	Total cost [1] per house	All houses		Total cost [1] per house	All houses
Group 1 houses—152:			Remodeled farm houses—12:		
Minimum	$4,076		Average	$522	
Maximum	10,170		Total		$6,236
Average	6,677		Duplex houses—20 families:		
Total		$1,014,992	Average	1,311	
Group 2 (cinder block)—80:			Total		26,219
Minimum	2,145		Apartments—30 families:		
Maximum	3,177		Average	1,920	
Average	2,674		Total		57,588
Total		213,943	Two-room guest house	1,935	1,935
K type—49:					
Average	2,742		Total cost[1]		1,445,277
Total		124,364			

[1] Additional cost data on houses built by the regular TVA construction organization appear in tables 41 to 46.

Table 45.—*Comparison of the three general classes of houses*

	Average of group 1	Type K–C	Types A, B, C, D
Cubic feet	15,265	9,600	9,442
Square feet of floors	1,094	662	807
Ratio	13.7	14.5	11.7
Job cost:			
Per house	$5,752	$2,162	$2,305
Per room	1,133	540	542
Per cubic foot	0.376	0.225	0.242
Per square foot floor	5.26	3.27	2.88

Table 46.—*Quantities and costs—cinder block houses*

	Type A	Type B	Type C	Type D	Type K–C
Quantity built	30	10	20	20	32
Number of stories	1	1	2	2	1
Number of rooms	4	4	4	5	4
Cubic feet	8,424	8,470	10,500	10,375	9,600
Square feet of floor	624	605	1,000	1,000	662
Per house:					
Direct cost	$1,735	$1,590	$2,430	$2,580	$1,973
Job cost	$1,904	$1,744	$2,663	$2,828	$2,162
Total cost	$2,209	$2,025	$3,090	$3,281	$2,506
Per room:					
Direct cost	$434	$397	$608	$516	$493
Job cost	$476	$436	$666	$565	$540
Total cost	$552	$506	$772	$643	$627
Per cubic foot:					
Direct cost	$0.206	$0.188	$0.231	$0.249	$0.205
Job cost	$0.226	$0.206	$0.254	$0.273	$0.225
Total cost	$0.262	$0.239	$0.294	$0.316	$0.261
Per square foot:					
Direct cost	$2.78	$2.63	$2.43	$2.58	$2.98
Job cost	$3.05	$2.88	$2.66	$2.83	$3.27
Total cost	$3.54	$3.34	$3.09	$3.28	$3.79
Ratio of cubic feet to square feet floor	13.5	14.0	10.5	10.4	14.5

Note.—Direct cost includes direct labor and materials only; job cost includes job burden, job overhead and medical services; total cost includes design and technical services and administration.

CIVIC GROUP

The character of the community center was dependent on the type of town that was to be developed. In the beginning it was anticipated that industries might be located at Norris, and with this in view a rather comprehensive scheme for a civic and business group was contemplated. It gradually became apparent that the prospects of industrial activity might be long deferred, and for this reason the plans were reduced in scope from time to time so as to be in scale with actual occupancy of the town. As the school was an indispensable element, attention was first focused on a school building.

Another element of the civic group that was early considered, and the first one on which construction was started, was an "agricultural building" intended as a food market which would be supplied, at least in part, by local farmes who would come in with produce and sell from their wagons in the rear of the building. At this time it was also thought that the inhabitants of Norris would engage in part-time agriculture to a considerable extent, and the building was intended to serve as a store for selling farm implements, seed, and other farm items. A building was actually constructed in the summer of 1934 which was intended for these purposes. This idea was

subsequently abandoned and substantially the same building, remodeled and added to, now serves as a drug store, food store, telephone exchange, and post office.

School building.

Plans for the school were studied in a general way in the early part of 1934 and were carried on actively in the summer of 1934. As there was a shortage of high-school facilities in the neighboring parts of the county, it was thought feasible to build the school so that it could be used in combination with the county (the county paying a share of the expenses), and on this basis the cost of reasonably good construction was more readily justified. It appeared that a capacity of at least 300 pupils would be required immediately, and that there would soon be a permanent use for a capacity of at least 400.

By this time the question of electric heating was prominent, and comparative studies were made to determine the feasibility of heating the school building electrically. There were proponents of both coal and electric heating; but in view of the interest of the Authority in electrical developments, it was decided to heat the building electrically.

On account of the location on a hillside slope, the school was designed two stories high at the front where the ground was low and one-story at the rear where the grade was approximately at the floor level of the upper story.

The arrangement of floors assured adequate fire exits for both levels. While a strictly fireproof construction was not adopted, it was highly fire-resistant. The first story was designed with a concrete slab floor resting on grade or fill, and the heating room and transformer vault were enclosed in fireproof concrete walls. A part of the lower story at the rear was dug into the hillside. The exterior walls above the footing and concrete walls were built of solid brick mostly 12 to 16 inches thick, and interior partitions, 8 inches thick, were of the same construction. Great care was taken with the workmanship of the walls, with the result that no trouble was experienced later with filtration of moisture from the outside, a trouble which is often found in brick walls without interior furring. The ceiling of the first floor was metal lath and plaster throughout. As fire protection was less essential at the top of the building, the ceiling of the second story was finished with insulation board. The floor construction of the second level consisted of heavy wood framing with a hardwood finished floor. The auditorium was built with steel columns and steel roof trusses. The steel trusses were also used for the clear span of the kindergarten room and the library. Over these trusses the roof construction consisted of 2-inch wood planks, and the remainder of the roof was of ordinary wood frame construction. The roof covering over the entire building was standard V crimped galvanized-iron roofing.

A sloping roof was adopted as being more in keeping with the residential characteristics of the town. A successful effort was made to keep the exterior treatment of the building plain but pleasing in proportion. The central windows at the front and all of the entrances were given an architectural treatment especially designed to

accent the points of interest without departing from the simplicity of the scheme as a whole. The fenestration employed large double-hung windows.

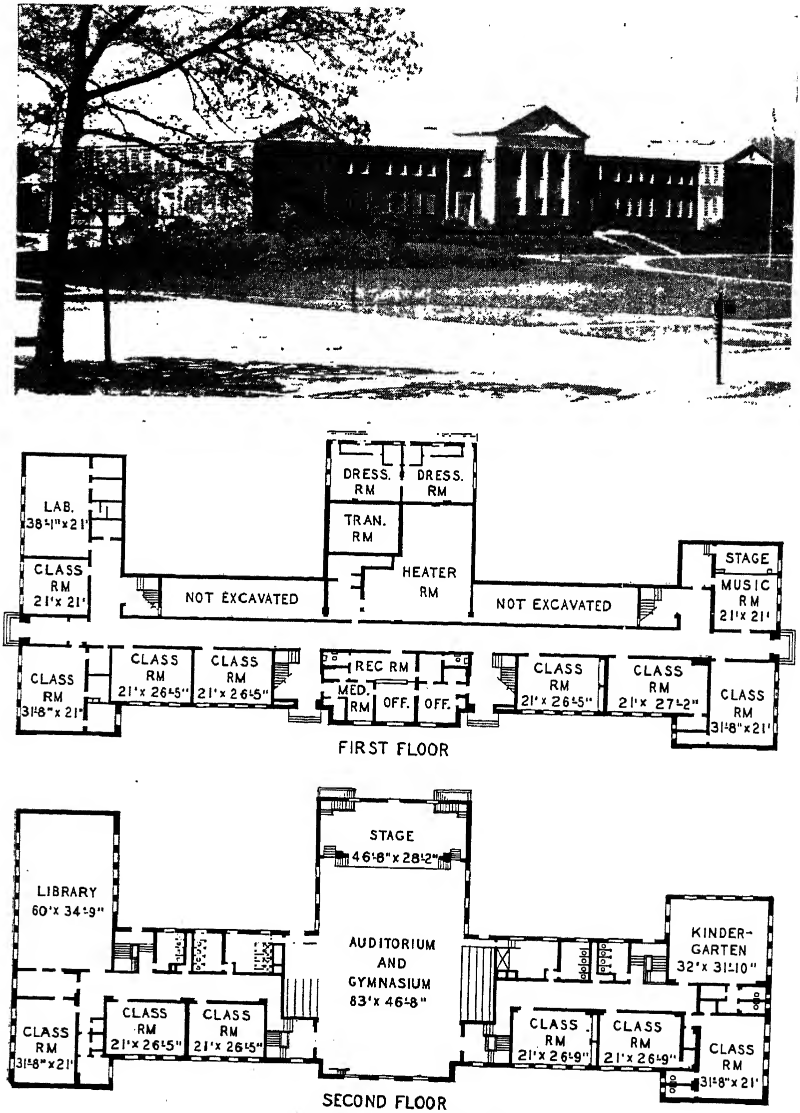

FIGURE 99.—*School building.*

Between October 1934 and February 1935 classes were held in seven K- and S-type houses from which the partitions had been temporarily omitted so as to form classrooms the full size of the house. Construction of the school building proceeded smoothly and it was occupied in February 1935.

Workshop.

The building which housed the training activities during the dam construction period was vacated when the space was needed for laboratory facilities. As there was no space available for a workshop in the school building, a supplementary structure was designed and built in the winter of 1935–36.

This building was placed a sufficient distance to the rear of the school building off the southeast edge of the play yard so that the noise would not be too objectionable. It was desired that this shop should exemplify a type of community facility that might be duplicated by other small modern communities. The type of construction was, therefore, limited to wood frame with board and batten walls and galvanized-iron roof, but the proportions and fenestration of the building were carefully studied in relation to the environment and harmony with the architectural forms used in the school building. Steel sash was used, spaced in a regular pattern, and the color treatment gave a subdued and satisfactory tone to the structure.

The building was equipped mainly for woodworking with machines transferred from the former shop. Some space was used for metal-working, and a paint shop was provided. A large canopy or shed at the rear of the building was built to give supplementary space. This space was later enclosed.

Store building and town office.

As previously stated, an agricultural building was planned as a part of the town. However, in view of the fact that agricultural possibilities in the immediate vicinity proved to be very limited, it was decided to change the use of the agricultural building by altering it, completing it in a different way, and adding to it a structure to house the town office. The design and construction of this building proceeded during the fall of 1934 and up to the summer of 1935.

The original "agricultural building" was extended to the rear, and the fenestration of the front was changed. The plans were changed to include a public toilet, drug store, food store, telephone exchange, and storage room. Later a design was made for installing the post-office between the food store and the telephone exchange. This work was carried out when the temporary post office in the community building was to be abandoned. Design of the interior appointments of the drug store and food store received careful study to exemplify modern practice carried out with due restraint, taking into consideration the small size and unpretentious character of the town and yet recognizing that these stores, particularly the drug store, would be exposed to a great deal of public inspection by tourists.

The building was of brick and wood construction with a concrete slab floor and asbestos shingle roof. A finished floor of asphalt tile was added to the food store and drug store, and the interior surfaces

and ceilings were given appropriate treatment for the two types of commercial enterprises, including all necessary store fixtures of modern design. The building is heated with an electric-blast heater system with a novel type of warm-air discharge below the ceiling and with baseboard type convection heaters. Modern lighting was installed in both stores.

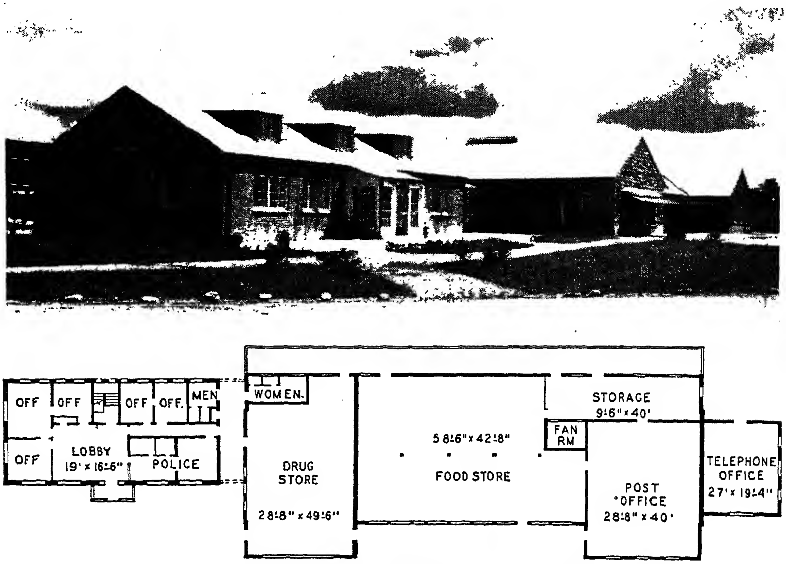

FIGURE 100.—*Store building and town office*

Construction work proceeded simultaneously on the addition extending to the northeast of the store building which provided office space for administration of the town and various activities in the reservoir area. This building was designed with brick walls and a precast integrally-colored cement slab-floor construction similar to that employed in the cinder-block houses. Interior surfaces of walls and ceilings were finished with plywood in architectural patterns. A considerable amount of office space was developed in the half story under the roof which was augmented by dormers at the front and back. This building also contains a public toilet. The offices are heated with electric radiators recessed in the walls.

A metal canopy in front of the store building united the two stores and the post office, and a stone terrace was laid the entire length of the offices. Both the drug store and second floor of the office building were later air conditioned.

Fire station.

The other building which has been included in this civic group is a combination fire station and firemen's living quarters southwest of the store building. The location was adopted partly for con-

venience in combining police service (which was established in the town office building) with fire service, as the man who would always be on duty at the police desk could also be available to drive the fire truck. The structure was improvised by moving one of the duplex houses, readjusting the two parts in relation to each other, and remodeling the entrances. A driveway was arranged permitting an easy turn into West Norris Road in either direction.

COMMERCIAL AND SERVICE STRUCTURES

Commercial structures.

An overnight tourists' camp consisting of 15 cabins was designed and built near the construction camp area. It was designed and constructed in lieu of another bunkhouse and was used for housing workmen during a considerable period. The cabins were provided with plumbing and kitchen equipment and screened porches. Since it was taken over as a supplement to the construction camp during the peak of employment, the tourists' camp was never fully developed as such during the construction period. However, after the major portion of construction was completed, the camp was used for tourists.

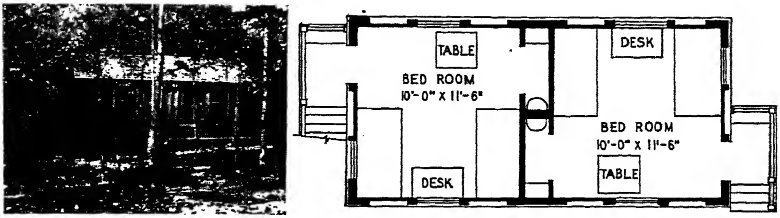

FIGURE 101.—*Typical cabin in tourist camp.*

At the junction of East Norris Road with the freeway a traffic intersection was especially designed to take care of the meeting of these two roads and the crossing of State Highway No. 61. It was also designed to accommodate a filling station on a central island between lanes of traffic. This is the only filling station on the freeway. It also is the only station close to the town. The lay-out of this intersection facilitates traffic flow and minimizes traffic hazards.

Dairy group.

A complete dairy lay-out consisting of a barn, milking house, and pasteurizing plant was constructed to furnish milk and butter for the village and camp and to serve as a working model showing progressive methods of producing and handling dairy products. It also provided a training center for TVA workers desiring to gain actual experience in these methods.

Laboratory buildings.

The building originally built as a tradeshop but later taken over for laboratory space was remodeled in the spring of 1935. The building is a large brick structure of mill construction with steel

framing for the roof. Large side windows and a clerestory are provided to insure ample light.

At the present time this building houses a hydraulic laboratory, a soil mechanics laboratory, a water analysis and silt laboratory, a minerals testing laboratory, and an office for field engineers who collect hydrological data in the Norris area.

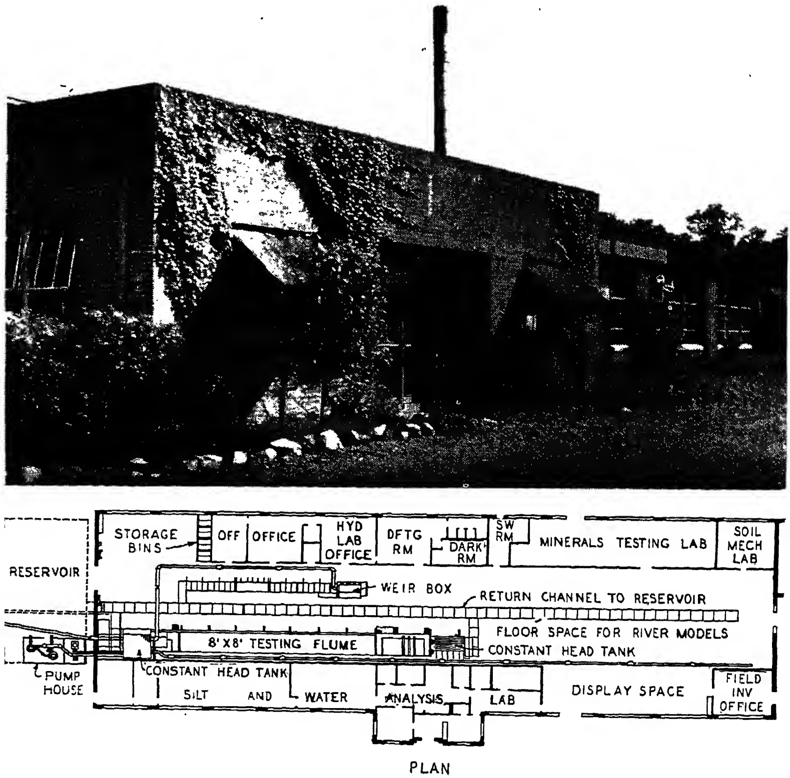

FIGURE 102.—*Laboratory building.*

A ceramics laboratory was built consisting of a research laboratory, an office, and a display room. The main part of the building is of wood construction while the L which extends toward the street is built of stone. This arrangement brings the building into satisfactory relationship with the houses on the opposite side of the street.

COSTS

It was hoped at first that the large scale and direct handling of operations might result in extraordinary economy. These hopes were nullified to a certain extent by loss of efficiency due to the

necessity of pushing construction work at the same time that major decisions and plans were taking form. Under Government procedure and established public policy, it was not possible to take advantage of bargain prices in materials nor was it the policy of the Authority to exploit labor as is sometimes done in private operations. However, when all of these circumstances are taken into account, the actual costs are not out of line with those of private operations of a similar character. The rapidly changing business conditions (see fig. 157) and changing outlook of the Authority in its initial stages resulted in a departure from the original premises on which the design of the town was undertaken, but the cost of reproducing the same values on a more deliberate basis would not be substantially different.

It is now generally recognized by public and private agencies in the United States and elsewhere that residential construction presents a set of stiff economic conditions that has not yet yielded to attempts at major improvement. The difficulties lie not alone in the housing but in the land improvements which are necessary to serve the housing, and which, in the ordinary course of gradual building up of a community, are largely absorbed under taxes and assessments and escape more or less unnoticed.

Conditions of labor policy, time limit, and elasticity of plans had committed the TVA to a program of building the houses for the most part by direct force account. The alternative of letting at least part of the work to private contractors was not overlooked.

After construction had started on the first of the group 1 houses and as soon as designs and specifications approached final form, competitive bids were publicly advertised and taken on six house types. The types on which bids were asked were representative of the group 1 houses, as they include extreme and intermediate sizes, and the average cost of these houses as built happens to check within a few dollars of the average of the entire group. The contract conditions bound the bidder to TVA wage scales, but not necessarily to N. R. A. code prices for materials. In the comparison it will be assumed that the bidder would have conformed to the N. R. A. provisions. A comparison of the actual job cost as built by TVA with the low-bid adjusted cost is shown in table 47.

TABLE 47.—*Comparison of actual cost with bid prices*

House type	Average of the bids submitted		Low bids as submitted		TVA job cost as built [2]
	Before adjustment [1]	After adjustment	Before adjustment [1]	After adjustment [2]	
D1	$4,878	$6,211	$4,402	$5,735	$5,486
D2	4,568	5,901	4,194	5,527	5,318
N2	4,666	5,999	4,055	5,388	5,155
32	4,453	5,786	4,098	5,431	5,246
41B	5,100	6,433	4,532	5,865	6,102
54	5,970	7,303	5,495	6,828	7,296
Total	29,635	37,633	26,776	34,774	34,603
Average of 6 types	4,939	6,272	4,463	5,796	5,767

[1] Bids adjusted to include work to be done by the Authority, materials to be furnished by the Authority and extras added after taking bids.
[2] Comparable figures.

TVA costs for design and administration are not included in these figures. They would have been approximately the same in either force-account or contract work.

As a further check on costs, a few of the houses, the filling station, the apartments, and the town office were assigned to a small, independent organization for construction. All materials and labor used by this organization were procured through regular TVA channels, but costs were kept separate in order that proper comparisons could be made. The quality of the work done by this organization was slightly below that of the regular TVA house construction. The costs were, in general, higher than those secured by the larger regular construction organization.

All Norris housing construction was virtually complete by June 30, 1935, and the statement of costs of construction of the town of Norris and the camp as of that date may be summarized approximately as shown in table 48.

TABLE 48.—*Cost data, June 30, 1935*

	Direct construction cost	Total cost (including design and overheads)
Clear. site, including landscape	$65,227	$82,987
Streets, signs, etc.	152,286	194,513
Service drives, garage courts	12,036	15,312
Path system	12,339	15,701
Water system	134,355	170,950
Sewer system	135,433	172,322
Storm drainage	5,819	7,402
Electricity	120,635	143,347
Total	638,130	802,534
Temporary fire station	75	95
Mess hall	35,202	44,787
Bunkhouses and dormitories	183,619	233,628
Community building	48,734	62,007
Total	267,630	340,517
Trade shop	38,918	49,516
Sawmill	2,032	2,584
Dairy barn	7,687	9,781
Milkshed	2,294	2,917
Horse barn	2,313	2,943
Creamery	14,003	17,820
Total	67,247	85,561
Remodel houses for school	3,719	4,731
Hotel (canceled)	438	557
Commercial building (canceled)	201	254
Total	4,358	5,542

	Direct construction cost	Total cost (including design and overheads)
School	$127,019	$161,616
Tourist camp	32,563	41,430
Tennis courts and athletic field	7,221	9,187
Service facilities	2,408	3,062
Plant yard and warehouses	9,377	11,934
Total	19,006	24,183
Town office and store building	62,464	79,330
Freeway filling station	13,888	17,639
Ceramics laboratory	18,309	23,252
Total	94,661	120,221
Garages (large groups)	19,350	24,618
Remodeled farm houses	4,921	6,236
Apartments (5 buildings)	45,259	57,588
Total	69,530	88,442
152 houses, group 1	797,714	1,014,992
One 2-room guest house	1,651	1,935
80 houses, cinder block group	168,146	213,943
49 houses, K type	97,924	124,364
10 duplex houses (20 families)	20,620	26,219
Total	288,341	366,461
Grand total	2,406,199	3,051,499

This statement does not include movable equipment and machinery, livestock, repairs, remodeling, or other maintenance work and developments incurred subsequent to June 30, 1935.

Additions and betterments to the town properties incurred between June 30, 1935, and June 30, 1938, and the inclusion of additional nearby rural electrification lines, marine equipment, and other operating equipment increased this investment to $3,887,984.33 as of June 30, 1938.

CAMP AND VILLAGE CHARGES TO NORRIS PROJECT

The Norris multiple-purpose hydraulic project was charged $918,999.50 for employee housing accommodations. This amount (including the total cost of construction and operation of the camp, and normal depreciation on the permanent town facilities during the construction period) is analyzed in table 49.

TABLE 49.—*Amount of cost of camp and village charged to the project*

	Income	Net expenses [1]	Net cost
Dormitories, community, and other buildings	$296,030.84	[2] $535,865.88	$239,835.04
Norris public school	5,541.79	98,568.95	93,027.16
Cafeteria, commissaries, barber shop, drug store, food store, creamery, and filling station	796,444.35	885,027.81	88,583.46
Utility services, streets, grounds, public health, and safety	62,995.45	170,421.98	107,426.53
Dairy, poultry, and truck farms	1,643.61	61,349.83	59,706.22
Employee training and recreation	21,954.74	273,478.53	251,523.79
Undistributed administrative costs to June 30, 1935 [3]		78,897.30	78,897.30
Total	1,184,610.78	2,103,610.28	918,999.50

[1] Included here are (1) all operation expenditures, (2) depreciation on buildings and equipment ($614,744.02 which includes the entire first cost of temporary camp facilities and normal depreciation during the construction period on the permanent town facilities), and (3) interdepartmental credits ($521,453.03) which represent charges to other departments and operations for use of buildings and roads, policing, electricity, water, and similar services.

[2] Includes all building depreciation for fiscal year 1935. In all other years such depreciation was either charged direct to various operations or distributed through rental charges thereto.

[3] Administrative costs subsequent to June 30, 1935, were distributed to the various operations.

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CHAPTER 5

CONSTRUCTION PLANT AND RIVER DIVERSION

As in the case of the design of the dam and powerhouse, the first preliminary construction plant and river diversion studies were conducted by the United States Bureau of Reclamation at Denver, Colo. However, final construction plant and river diversion studies and designs were conducted by the Authority. The design, construction, and operation of the temporary facilities used in construction work and the temporary facilities used in the river diversion operation are described in this chapter.

CONSTRUCTION PLANT

The principal construction problem at Norris Dam was to manufacture and place approximately 1,000,000 cubic yards of concrete. This concrete had to be placed at such extreme elevations as the bottom of the draft tube excavation, which was at elevation 792, and the top of the roadway over the dam, which was at elevation 1,061, and within an area defined by the width of the base of the dam and apron, which was 423 feet, and the length of the crest of the dam, which was 1,570 feet.

The construction schedule called for completion of the major concrete structure in 18 months. Early laboratory aggregate and concrete investigations showed that concrete made from aggregate crushed from dolomite rock, which was available in unlimited quantities close to the dam site, had the desirable strength, durability, and thermal properties to be satisfactory for use in the dam. It was possible, since the quarry was close to the dam axis, to lay out an aggregate manufacturing and concrete mixing plant to provide for continuous operation from the quarry to the dam.

Cement for the dam was unloaded from freight cars at Coal Creek, Tenn., by portable pumps into a 6,000-barrel storage silo. Truck-trailer hauling units then delivered the cement as needed to the dam where it was dumped into a hopper, and two stationary pumps delivered it to the bins in the mixer plant or to a 6,000-barrel storage silo.

The principal units of the construction plant consisted of a primary crusher at the quarry, discharging onto a belt conveyor which carried the aggregate across a valley to a secondary crusher. The secondary crusher discharged onto another belt conveyor that carried the aggregate onto a screening structure where the rock was screened out into the four large sizes of coarse aggregate and placed in storage piles. The remaining small sizes and surplus from the large sizes were carried to the sand plant, where two sizes of fine aggregate were manufactured. The average designed capacity of the aggregate manufacturing plant was 300 tons per hour, but actual continuous production reached 350 tons per hour.

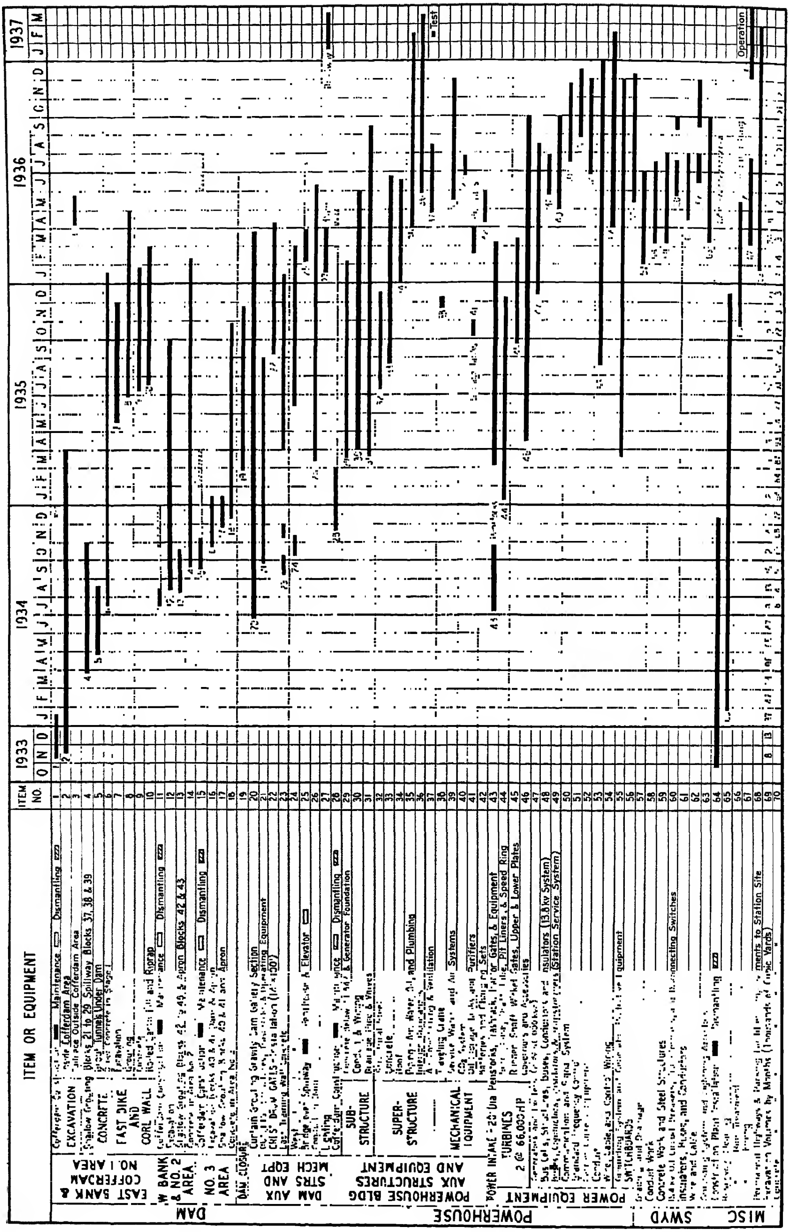


FIGURE 103.—*Construction schedule.*

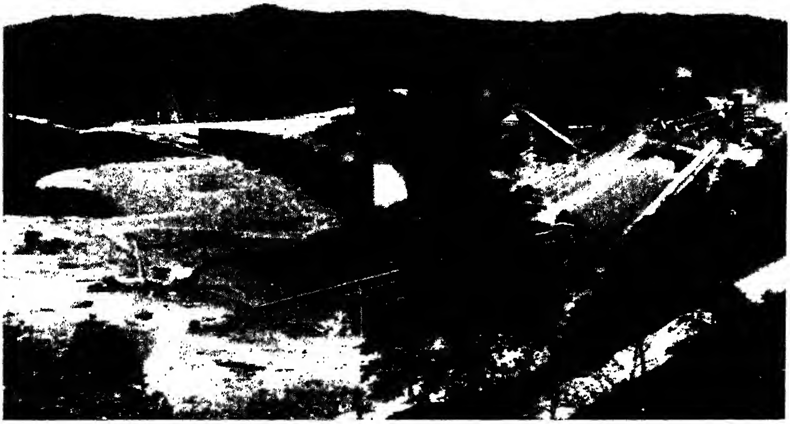


FIGURE 104.—*General view of plant.*

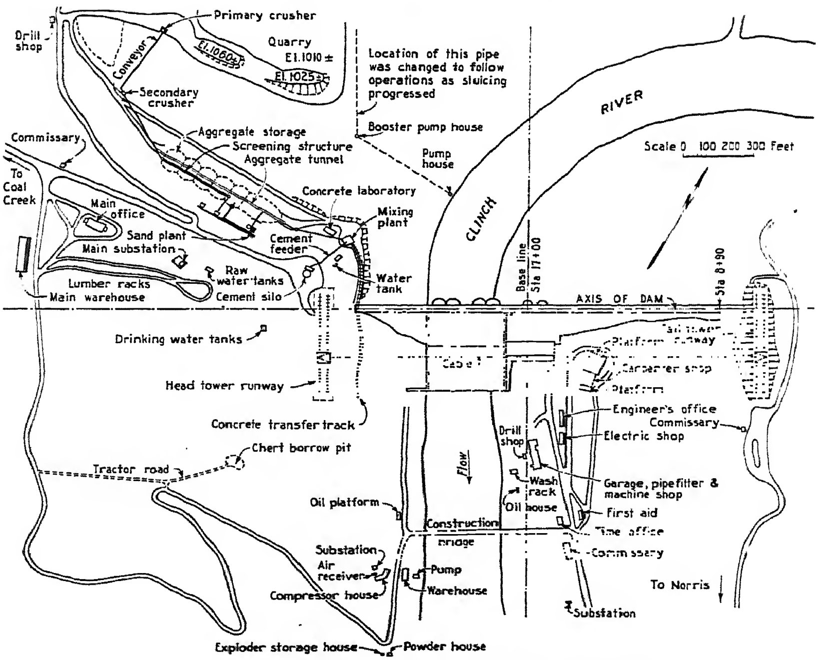


FIGURE 105.—*General lay-out of construction plant.*

The mixer plant was located on the west bank of the river, level with the top of the dam and approximately 300 feet upstream from the axis. A belt conveyor in a tunnel under the aggregate storage piles conveyed the aggregate to the mixer plant. Bins for cement and aggregate were located at the top of the steel mixing plant structure. From the storage bins the aggregate and cement flowed by gravity into batch-weighing hoppers and from there the material was discharged into a conical steel hopper that fed any one of the three

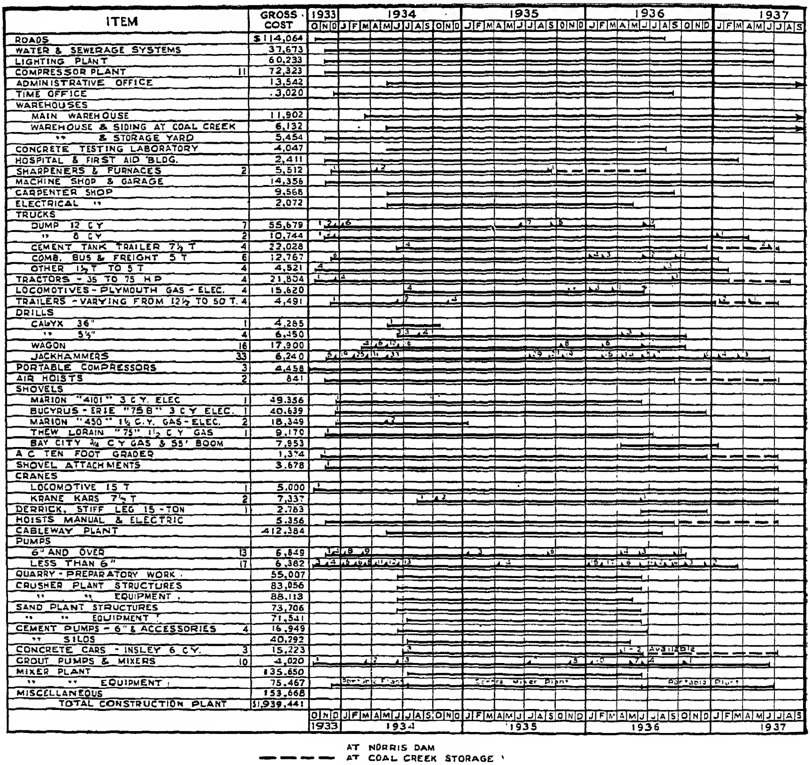


FIGURE 106.—*Construction plant equipment schedule.*

mixers. Concrete from the mixers was discharged through a conical hopper into transfer cars which hauled the concrete to a point directly under one of the two cableways. There the concrete was discharged into a bottom-dump cableway bucket. The cableway finally transported the loaded buckets to the forms where the concrete was dumped. The cableways were designed to handle a maximum of 75,000 cubic yards of concrete per month, but actually placed a maximum of 92,782 cubic yards in 1 month.

Other plant facilities such as warehouses and shops; compressed air, raw-water, and electric power systems; and the roads were laid out and designed to conform to the needs of the concreting operations.

The design and selection of the construction plant was made by a staff of engineers under the supervision of the construction plant engineer and direction of the construction engineer. The work was performed in close collaboration with the construction superintendent and with the advice of a construction consultant. This program was in accordance with the policy of the board of directors to perform the construction work by the Authority's forces.

The source of aggregate supply was a problem around which the preliminary investigations were centered, and in this connection two general sources were immediately obvious: first, the purchase of aggregates from existing aggregate producers, including both natural and manufactured aggregates; and second, the manufacture of aggregates from the dolomite rock by means of a plant set up and operated by the Authority. The most promising of the several existing plants were studied, and the pertinent facts in regard to the plants in the immediate vicinity of Knoxville were collected. Several other sources were named, but many of these were eliminated from further consideration by high freight rates.

The proposed quarry site offered the best possibilities because of proximity to the job, the almost unlimited quantity available, and the known excellent quality as large aggregate. The quarry site chosen was just upstream from the dam on the west bank of the river and offered a favorable possibility of locating the primary crusher at the edge of the quarry, thus reducing the handling of quarried material to a minimum. The quality of manufactured fine aggregate, particularly that manufactured from dolomite, or the limestone family, was doubtful because of some recent observations of deterioration in concrete structures containing sand manufactured from limestone. As a result of this doubt, a program of investigation was carried out. Samples of material from the outcropping ledges of rock at the dam site and samples from several of the existing producers were tested. The results of these tests removed all doubt as to the quality of the existing dolomite as fine aggregate for use in concrete.

The fact that dolomite rock was suitable for aggregate led to some very definite conclusions, all of which favored the use of dolomite rock for the production of concrete. These conclusions are summarized as follows:

1. Only two existing producers within reasonable freight rate range had plant capacities and available material sufficient to produce the quantity necessary.

2. Neither of these producers had a plant already set up for processing the aggregate, as desired by the Authority. The existing plants would have required considerable revision in order to have been able to produce the desired product.

3. Transporting all aggregate to the job from an outside source would practically have necessitated the construction of a railroad

to the site. Estimates of transportation costs indicated that, for movement of all aggregate from an outside source, the total cost of all material delivered by railroad to the job would have been approximately $570,000 cheaper than delivery by highway.

4. Transportation of all aggregates to the job from an outside source would have resulted in an estimated increase in total cost of all materials delivered to the job of approximately $1,170,000.

5. Estimated delivered costs of aggregates from an outside source by the cheapest method of transportation were $1.490 per ton for fine aggregate and $1.391 per ton for coarse aggregate. Estimated delivered costs of aggregates manufactured on the job from dolomite rock were 80 cents per ton for fine aggregate (actual job cost $1.01) and for coarse aggregate 70 cents per ton (actual job cost 49.7 cents).

6. Close control of aggregate gradation necessary for the production of high quality concrete and for proper cement economy could be more easily obtained in a plant operated by the Authority.

7. Tests indicated that aggregate produced from dolomite rock was superior to the natural aggregate available.

8. Direct control of the production of the aggregate by the Authority was desirable for many reasons, particularly from a standpoint of coordinating the other phases of the construction program with aggregate production.

It was obvious from these conclusions that the most dependable aggregate supply at the lowest cost and of the highest quality could best be obtained by manufacturing aggregates from the rock available at the site. Experience proved this to be true in every respect.

Methods.

The general method of procedure followed in quarrying was to develop a quarry face running east and west parallel to the ridge along which the quarry was located, then to carry the face back into the hill by drilling and blasting. Rock usually broke at stratification planes so that little work in addition to normal quarry operations was needed to provide a relatively smooth floor on which to work.

The first floor on which operations were undertaken was approximately at elevation 1,065. On this floor operations followed a bedding plane which sloped slightly upward in the north or upstream direction, so that haulage from the quarry face to the primary crusher was somewhat downhill. Above this floor drilling and blasting was done in 30-foot benches, leaving ledges with a minimum width of 12 feet for safety. Approximately 1,240,000 tons of rock were quarried above this level.

In quarrying below elevation 1,065, operations were carried on successively on two lower floors. The first lower floor varied in elevation between 1,019 at the east end and 1,045 at the west end. This floor was reached by a ramp with a maximum grade of minus 12 percent proceeding eastward from the primary crusher. Operation on this floor was started at a point midway between the primary crusher and the southeast corner, proceeding north to an average of 20 feet from the previously established face, and west 250 feet beyond the primary crusher. A space 90 feet wide was left in front of the primary crusher to permit maneuvering of trucks.

A second lower floor varied in elevation between 980 at the east end and elevation 1,018 at the west end. This floor was reached by an extension of the ramp from the primary crusher to the first floor level. Operations on this level were started at the southeast corner and proceeded to the north and west faces until no more material was needed.

As a precaution against possible injury to the primary crusher caused by vibrations set up by blasting below the original quarry floor, the primary crusher was isolated from the surrounding area by a line of closely drilled holes. Isolation of the crusher area successfully protected the crusher and permitted completion of quarry operations on the lower levels without reducing blasting efficiency.

Stripping.

Due to the steep slope of the original surface of the quarry and the relatively small depth of overburden, the most economical method of stripping this overburden was by hydraulic methods. Hydraulic

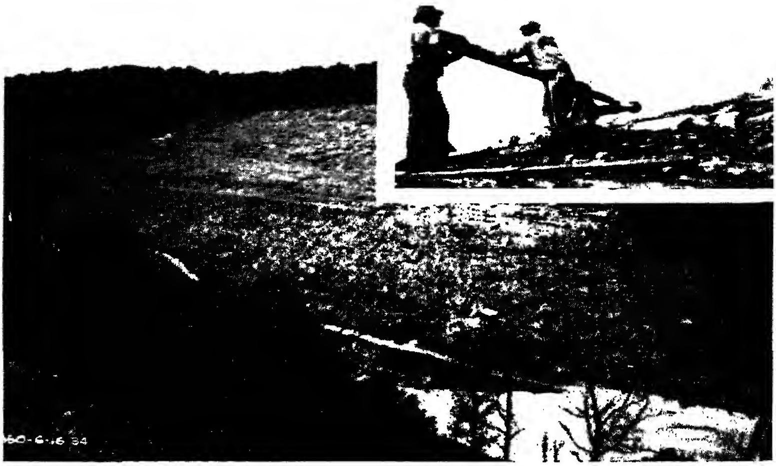


FIGURE 107.—*Sluicing operations.*

monitors with 2½-inch nozzles were used in this work. Water was supplied through 12-inch pipe by two centrifugal pumps working in tandem and one booster pump delivering 1,750 gallons per minute at nozzle pressures varying between 100 and 150 pounds per square inch. A total of approximately 90,000 cubic yards of overburden was removed by this method between January 11, 1934, and June 23, 1934. At the end of the sluicing operation, the entire system was dismantled and salvaged.

The material removed was washed down into the ravine leading to the river just upstream from the dam. In order to keep this material from clogging the river diversion channel and otherwise interfering with construction operations, a series of light timber baffles was built at the mouth of the gully to retard the flow and allow the overburden to settle out.

Although sluicing was successful in removing most of the overburden, drilling and blasting were necessary to clear away top strata of unsound rock together with the remaining clay. The top strata were characterized by the extreme irregularity of the rock surface with deep crevices and protruding pinnacles. The mixture of clay and unsound rock overlying the good rock was wasted. These secondary stripping operations were performed considerably in advance of the quarrying operation in order to avoid mixing the good rock with the stripped material.

Removal of clay without blasting was tried and used over small areas, but was generally more expensive. Thorough cleaning was expensive and often required wasting appreciable quantities of good rock; but the advantage of clean rock in producing concrete of high quality and in improving crushing, screening, and washing plant operations more than outweighed the extra cost.

Quantities and a summary of costs for developing the quarry face, including the hydraulic stripping operation, are given in table 50.

TABLE 50.—*Quantities and costs developing quarry face*

	Quantity	Unit cost	Amount
	Cubic yards		
Preliminary investigations			$2, 931. 93
Clearing			3, 679. 11
Hydraulic stripping	89, 258	$0. 485	43, 306. 16
Excavation of weathered rock to establish working face	12, 682	. 281	3, 560. 43
Miscellaneous expense including dismantling			1, 529. 76
Total quarry development			55, 007. 39

NOTE: These costs are considered to be the plants first cost and are not to be confused with the secondary stripping cost of $53,675.83 which is an operating charge for the removal of 178,231 tons o f weathered rock and muck.

Drilling.

The rock in the quarry is classified as Knox dolomite and contains approximately 5 percent silica. It ranks above marble and sandstone but below trap in toughness. Its abrasive qualities are lower than granite and trap but above most limestones, dolomite, slate, and shale. The dolomite lay in nearly horizontal strata varying from 6 inches to 6 feet in thickness, usually separated by very thin seams containing negligible amounts of undesirable material. No noticeable variations in quality of rock at various elevations of the quarry could be noticed by visual inspection; however, the variations in wear in the hammer mills indicated differences in abrasiveness. Chemical analyses of composite aggregate samples representing monthly products showed some changes in silica content but very small changes in other constituents. Metal wear in hammer mills followed closely the changes in silica content of the dolomite shown by the chemical analyses. Uniformity of the deposits eliminated all necessity for selective quarrying.

Drilling for primary blasting was done by pneumatic wagon drills. There were 12 of these drills of four different makes in service during the period of principal activity. The blast holes were drilled on approximately 8½-by- 6-foot centers and were approximately 30 feet deep. For the relatively small amount of secondary blasting, pneumatic drills of the jackhammer type were used.

Compressed air was supplied to the quarry area from the central compressor plant situated about 3,000 feet away.

Because of clay and rock being left on the benches after blasting, much hand mucking was necessary. In general, the mucking was done by a labor crew working behind the drills.

FIGURE 108.—*Wagon drills in operation.*

A typical drilling crew for standard forged bits with 12 drills in operation is given in table 51.

TABLE 51.—*Typical drilling crew—12 drills*

Crew	Classification	Number	Hourly rate	Total hourly earnings
Drill sharpeners	Blacksmith		$1.00	$1.00
	Blacksmith helper		.60	1.20
Drilling	Drill operator		.75	9.00
	Drill helper		.45	5.40
Nipping	Laborer [1]		.45	.90
Cribbing up and moving drills	do		.45	.90
Foreman	Drill foreman		.80	.80
Water boy	Laborer		.45	.45
		35		19.70

[1] Normal operation. For high quarry face as many as 6 were used.

When quarrying was in progress above the elevation 1,065 floor, the quarry face approached the height of 100 feet, and there were, generally, three benches on which drilling and blasting were done. Moving the wagon drills from one location to another presented some difficulties since each drill weighs about 1,200 pounds. A method frequently employed utilized a cable running through a snatch block anchored at the top of the quarry face to hoist the drill, while another

line was used to hold the drill away from the rock face of the quarry Motive power was either a double drum hoist mounted on a tractor or two quarry trucks.

A certain amount of cribbing was required under the wagon drills in order to hold them in the proper position for drilling. This work was performed by laborers attached to the drilling crew.

Drill performance was observed carefully over a period of 6 months of the heaviest drilling operation, during which time both forged and detachable drill bits were used. The average performance and cost[1] of the 12 drills operating during that period are shown in table 52.

TABLE 52.—*Drilling operations, average performance and cost, 12 drills*

Performance:		
Total possible working hours		32, 895. 5
Total hours drill crews mucking		18, 434. 5
Gross drill hours		14, 461. 0
Delay—hours:		
Hung steel	522. 7	
Moving drills	2, 711. 3	
Miscellaneous	1, 028. 2	
		4, 262. 2
Net drilling hours		10, 198. 8
Total linear feet drilled		305, 808. 0
Feet per gross operating hour		21. 1
Feet per net operating hour		30. 0
Tons of rock moved		949, 233. 0
Tons per foot of hole		3. 1
Lost footage		11, 269. 6
Cost:		
Total operating labor		$25, 058. 23
Total repair labor	$1, 055. 46	
Total repair material	2, 176. 85	
Machine time charges	60. 59	
Total repair cost		3, 292. 90
Depreciation		1, 615. 51
Total cost		$29, 966. 64
Cost per gross operating hour		2. 070
Cost per linear foot		. 098
Cost of drilling per ton		. 0315

In the preceding table operating labor includes wages of drill crew personnel when on drilling operation only, the time on mucking being excluded. Repair labor and material does not include any shop or warehouse overhead. For this purpose, depreciation was based on an estimated life per drill of 10,000 operating hours.

Except for a 3-month period in which a special field test was made on detachable drill bits, during which time the latter were used exclusively, all drilling was done with standard 1¼-inch round hollow forged drill steel in lengths of 10 feet, 20 feet, and 30 feet. The bits were gaged 2½ inches, 2¼ inches, and 2 inches, respectively.

A drill sharpening shop was set up about 900 feet from the center of the quarry face at the west end of the ravine in which the quarry

[1] As determined by special studies.

was located. One blacksmith and two helpers were the standard crew who operated the drill sharpening shop.

Handling the drill steel between the quarry face and the drill sharpening shop required from two to six men and a truck or team and wagon. This phase of work is referred to as "nipping," and the work varied with the accessibility of the area in which drilling was done.

During the period from November 1, 1934, to April 30, 1935, a comparative study of the operation of various rock drill bits was carried on at the quarry. The purpose was to determine the most economical and satisfactory type of bit to use for drilling rock for the production of concrete aggregate for the dam. It was also to be used as a basis for making a decision as to the type of bit to use on various other projects of the Authority.

Three different makes of bits furnished by individual manufacturers were investigated, as well as the conventional forged bit. Because of the very close comparative costs of drilling obtained with three types of detachable bits, the varying conditions under which each type was investigated, and the amount of experimenting done on each test, the justification to recommend one type as being superior to another was not apparent. Since these tests were made there undoubtedly have been improvements in all types, and the results might be different from those obtained should the test be run at this date. However, from these tests one conclusion was reached: that the detachable bit was more economical for quarries such as the Norris quarry and in most locations where the transportation and nipping costs are an important factor.

Blasting.

Dynamite and electric blasting caps were used exclusively in the quarry blasting operations. Explosives were delivered by vendor's truck to the magazine on the job as needed. Two magazines, one for dynamite and the other for detonators, were located on top of the hill on the west bank of the river about 2,000 feet downstream from the quarry and 600 feet downstream from the main warehouse. Dynamite and caps were taken from the magazines in quantities sufficient only for immediate shooting, and cartridges to be used for detonators were primed in an isolated location as near the hole to be fired as safety would permit. Most of the holes were fired from 110-volt, alternating-current circuit provided for that purpose. However, in some instances holes were fired from portable electric blasting machines. All accepted safety precautions for the handling of explosives were rigidly enforced. As a result, no fatalities and only one serious accident occurred in the quarry in connection with the use of explosives.

Primary blasting in the quarry was in general kept fairly light in order not to run any risks of disturbing the foundation of the dam. The heaviest single shot used required 5,300 pounds of dynamite. The next heaviest shots were 4,300 pounds and 4,000 pounds respectively, and there were only a few shots using as much as 3,000 pounds. Seismographic investigations to determine the effect of quarry blasting on the rock foundation of the dam gave no indication that the shots were creating dangerous conditions. It was found that with the spacing of the blast holes described and with the explo-

sives used, the quarry rock broke up to such an extent that only a little secondary drilling and blasting were necessary.

A special study of the cost of blasting material only was made for quarrying 1,041,899 tons of rock during 6 representative months of operation. The results of this special study are shown in table 53.

TABLE 53.—*Quantities and costs, blasting material*

	Quantity used	Total cost	Cost per ton of rock quarried
Dynamite .... pounds..	333,950	$32,101.87	$0.031
Exploders	61,925	3,156.87	.003
Blasting wire	....	838.58	.0008
Total	....	36,097.32	.0348

The total cost for quarry blasting supplies was $79,183.79. These supplies were used to blast 2,283,419 tons of rock, of which 178,231 tons were wasted and 2,105,188 tons were usable. The rock was used in the following manner:

	Tons
Delivered to primary crusher	2,079,772
Used for riprap	23,332
Used for flood protection	2,084
Total	[1] 2,105,188

[1] This quantity is useable rock only and does not include 178,231 tons of muck and waste.

Loading.

The quarried rock as it fell on the quarry floor was reworked with jackhammers to less than 42 inches, the maximum size which could enter the primary crusher. This rock was then loaded for hauling to the primary crusher by two all-electric shovels, both of which were essentially the same in size and type although manufactured by two different companies. Table 54 lists the pertinent data concerning each shovel.

TABLE 54.—*Data on quarry shovels*

	Bucyrus	Marion
Manufacturer	Bucyrus-Erie Co	Marion Steam Shovel Co.
Type and serial number	75-B, 11595	4101, 6681.
Shipping weight .... pounds..	210,700	209,000.
General specifications:		
Dipper .... cubic yards..	3 (spare—2½ yards)	3 (spare—3 yards).
Boom .... feet..	29	31.
Dipper handle .... do..	20	21.33.
Mounting	Crawler	Crawler.
Swing	Full revolving	Full revolving.
Power rating	75 horsepower all electric	75 horsepower all electric.
Attachments	None	None.[1]
Purchase price f. o. b. factory	$36,725.00	$38,250.00
Freight	997.74	1,141.51
Assembly	[2] 620.42	[2] 2,147.96
Total	38,343.16	41,539.47

[1] A 70-foot boom and 2-cubic yard dragline bucket were borrowed from Wheeler Dam to clean out the debris in connection with a slide under the cableway head towers.

[2] In connection with the assembly costs, the Marion was transported down the hill on the west bank of the river and across the river before being assembled, while the Bucyrus was assembled on top of the hill on the west bank.

From November 1933, when these shovels were first received on the job, until June 1934, when they were moved to the quarry, both shovels were used for dam foundation excavation on the east bank and in the east cofferdam. They were also used on other work when they were not needed in the quarry. This included repair work in connection with a cableway head tower cave-in, grading of approach road to the top of the dam, and dismantling of the construction plant.

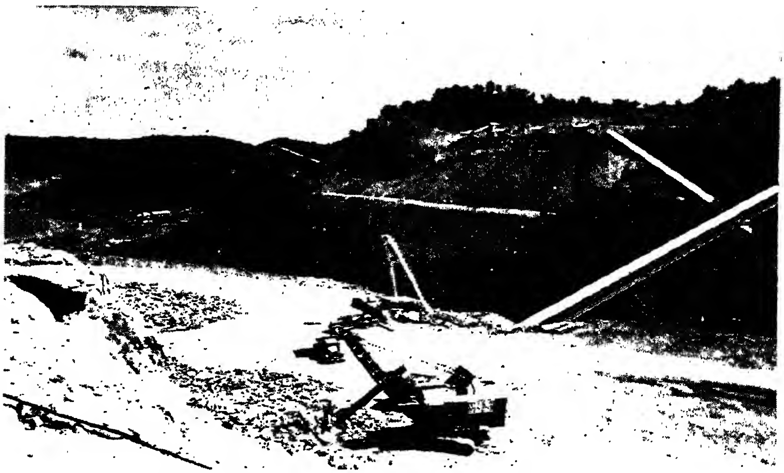


FIGURE 109.—*Shovels and trucks working in the quarry.*

The two shovels worked independently in locations widely enough separated to avoid interference from movement of trucks to and from the shovel. Each shovel was usually served by a 12-yard dump truck without delay as the number of trucks was sufficient to keep the shovels running full time. Each truck backed to the shovel to get its load while the shovel made a swing varying from 90° to 180°. As a rule, from four to five dipper loads were required to fill a truck. After each quarry "shot" the shovels concentrated on loading out of the large pile of rock, making no attempt to clean up. As the shovels moved into the large piles, a tractor equipped with a bulldozer followed and concentrated the scattered material into convenient piles for handling by the shovels. One such tractor was needed full time for this work.

As much as practicable during the period when secondary stripping was in progress, one shovel concentrated on loading the stripped material. This method of operation eliminated the possibility of stripped material getting into the primary crusher and being made into concrete aggregate.

Power for the operation of these shovels was supplied by a 2,300-volt line. From the west end of the quarry, power was carried directly to the shovels through rubber-insulated flexible cables which were moved about to conform to movements of the shovel. Special

precautions were taken to protect these cables from damage by truck, tractors, and the shovels themselves.

When operations were on a basis of four 5½-hour shifts, 1 hour was taken off at noon and another at 6 p. m. for servicing equipment. These periods were also utilized for the heavier blasting.

The standard shovel operating crew consisted of the following personnel:

	Per hour
1 operator	$1.50
1 oiler	.75
1 pit man	.45

In addition, a foreman working on 8-hour shifts at $1 per hour supervised both shovels.

Performance of both shovels during the period of maximum activity in the quarry, June 13, 1934, to October 1, 1935, is shown in table 55.

TABLE 55.—*Performance of quarry shovels during period of maximum activity—June 1934 to October 1935*

	Bucyrus	Marion
Time (hours):		
Elapsed time	11,080	10,720
Out of service, no operator	−2,743	−2,930
Gross operating time	8,337	7,790
Delays—mechanical, idle, and miscellaneous	1,611	1,550
Net operating time	6,726	6,240
Production:		
Tons	1,061,040	916.998
Cubic yards, loose	785,956	679,258
Loads	72,774	62,894
Tons per net hour	157.8	147.0
Cubic yards per net hour	116.9	108.9
Loads per net hour	10.8	10.1
Tons per load	14.6	14.6
Cubic yards per load	10.8	10.8

The net operating hours for both shovels for all work between December 1, 1933, and October 1, 1935, is summarized below:

	Bucyrus	Marion
Foundation excavation	2,141	1,444
Quarry excavation	6,726	6,240
Miscellaneous excavation	469	1,526
Total (all work)	9,336	9,210

Hauling.

Four White and three Hug dump trucks moved 2,283,419 tons of rock and muck from the quarry to either the primary crusher or to spoil. During the first 6 months of 1934 these trucks were used to haul rock and muck from the dam foundation. Following this excavation, they worked in the quarry.

These trucks had a 12-cubic-yard capacity and were equipped with Boulder Dam type dump bodies and dual telescoping hoist. They had tandem dual rear wheels and driving axles, providing a total of eight driving wheels.

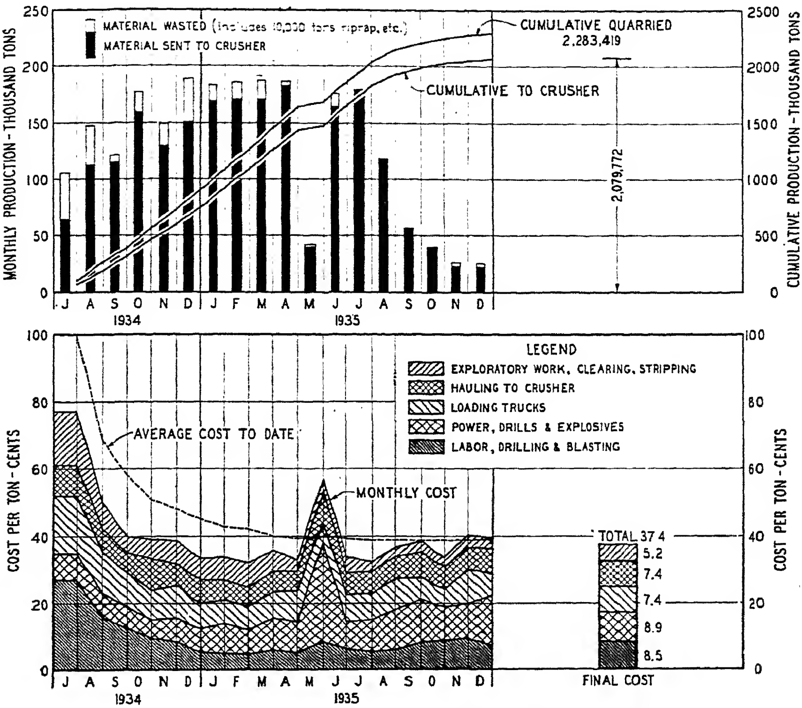


FIGURE 110.—*Quarry operation—Quantities and costs.*

Each truck was provided with a special auxiliary transmission providing 12 speeds forward and 3 reverse on the Hugs, and 10 speeds forward and 2 reverse on the Whites. The Hugs had a 175½-inch wheel base and the Whites a 185-inch, measuring from the center of the front wheels to the center of the connecting tube between the tandem rear axles. The gross rated capacity of the Hug trucks was 52,000 pounds and of the Whites was 60,000 pounds.

Both 10.50- by 24-inch, 12-ply, 5,200-pound-capacity tires requiring 75 pounds air pressure; and 10.00- by 44-inch, heavy-duty, 16-ply, 6,000-pound-capacity tires requiring 100 pounds air pressure were used by these trucks. Experience proved that the added unit cost per tire, which was approximately 50 percent, of the 16-ply heavy-duty tire over the 12-ply lower pressure tire was justified under these circumstances of operation.

Very little work was necessary, other than regular clean-up, to make the three quarry floors suitable for truck traffic. Each floor was formed by a relatively smooth natural bedding plane of rock and sloped between 2 and 5 percent downward from northwest to southeast. On the main quarry floor the haul was always slightly downhill to the primary crusher. The ramp to the two lower floors had a maximum upgrade slope to the primary crusher of about 12 percent.

These trucks were loaded to an average of about 14.5 tons, or about 10.8 cubic yards of loose rock. Dumping at the crusher was always by signal from the crusher operator. This occasionally resulted in some delay but was practiced consistently in order to avoid choking the crusher. The average time, less delays, incidental to a complete trip of the truck from the shovel to the crusher and return for the main floor, a distance of about 390 feet, was about 5.6 minutes; for the first lower floor, a distance of about 850 feet, the time was about 8.0 minutes; and for the second lower floor, a haul of an average distance of 1,250 feet, the time was 9.4 minutes.

Operating data showing the performance of these trucks between January 1, 1934, and October 20, 1935, the period of principal activity in the quarry, are given in table 56:

TABLE 56.—*Performance of quarry trucks during period of maximum activity—January 1934 to October 1935*

Period of activity:	
Truck months operated in quarry	97
Truck months operated in foundation, cableway cave-in, etc	33.3
Total	130.3
Possible hours of operation	69,486
Out of service and repairs	18,263
Gross operating hours	51,223
Miscellaneous field delays	2,789
Net operating hours—Quarry, 33,565; other, 14,869	48,434
Production in quarry:	
Loads rock	137,512
Loads muck	12,222
Tons rock—Loads by 14.58	2,004,900
Tons muck—Loads by 14.58	178,230
Cubic yards solid in place—Loads by 6.5	973,200
Cubic yards loose—Loads by 10.8	1,617,100
Total loads per net operating hour	4.46
Fuel used:	
Gallons gasoline	131,704
Gallons per net operating hour	2.72
Quarts cylinder oil	47,132
Quarts per net operating hour	0.975

On the whole, both makes of trucks gave good service with a normal amount of repair work. During 1934 the frames directly behind the cabs of all trucks required strengthening. The bodies showed signs of weakness in the joints between the sides and bottom. As a result the manufacturer changed the body design to conform with job requirements.

Spare assembled units were kept at the garage to facilitate repairs. All trucks except one underwent major overhauls in the spring of 1935, just prior to the opening of the first lift that required a heavy pull up an incline road. Inasmuch as the quarry was 1½ miles from the garage, many minor repairs and adjustments were made at the quarry by mechanics sent from the garage. Fuel was brought to the truck while on duty by the gasoline tank truck, and lubrication was taken care of by roving crews of oilers.

The quarry operating cost of these trucks as determined by a special study covering approximately 90 percent of their operation amounted to $133,910.54. The unit cost is summarized below:

	Cost per unit
Net operating hour	$3.990
Load	.894
Ton	.061
Cubic yard in place	.138
Cubic yard loose	.083

Estimated depreciation, based on 10,000 net operating hours per truck, results in a slightly higher depreciation for the Whites—$0.819 per hour as compared to $0.750 for the Hugs. The operating costs for labor and materials are based on the following basic rates:

	Unit	Cost
Gasoline, average	Gallon	$0.112
Oil, average	do	.262
Drivers	Hour	.75
Mechanics	do	1.00
Mechanics' helpers	do	.60
Oilers and greasers	do	.45

Operation—cost.

The cost for quarrying 2,079,772 tons of rock is given in table 57.

TABLE 57.—*Analysis of quarrying costs*

	Total cost	Cost per ton	Total cost per ton
Preliminary work:			
Quarry development and stripping	$107,629.85	$0.052	$0.052
Drilling and blasting:			
Drill operation:			
Labor and expense	142,066.07	.068	
Repairs to drills	10,914.36	.005	
Total drill operation			.073
Drill steel:			
First cost	10,315.78	.005	
Sharpening and transporting	29,550.31	.014	
Total drill steel			.019
Air and power:			
Compressor and electric plants	55,191.23	.027	.027
Blasting:			
Labor	35,248.99	.017	
Explosives and accessories	79,183.79	.038	
Total blasting			.055
Loading:			
Labor and expense	154,380.25	.074	.074
Hauling:			
Labor and expense	153,284.49	.074	.074
Total cost	777,765.12		.374

AGGREGATE MANUFACTURING PLANT

Decision to produce all of the concrete aggregate from rock found near the dam site was the most important in many respects of the several major considerations in connection with the construction of this project. After the decision was reached, a program of design and selection was inaugurated for the purpose of arriving at the best plan for handling the problems.

Because quality of concrete and efficiency of its production and placement were largely dependent on quality of aggregates, efforts were concentrated on maintaining accurate control of these products. The crushing and original aggregate screening plant was designed to fulfill the needs of concrete production up to 3,000 cubic yards per day, which would have required somewhat more than 300 tons of rock to the primary crusher per hour. With concrete production approaching 4,000 cubic yards per day, the load on the screening plant had to be increased accordingly; and as a result, screening equipment was overloaded and clean separation of the various sizes was extremely difficult even under the most favorable conditions. The constant load on the screens resulted in reasonable uniformity of products, and this condition rather than clean operation was necessarily relied upon in proportioning concrete mixes. During wet weather production rates were of necessity curtailed; and on a few occasions production was suspended entirely until dry crusher feed, or wet feed free from quarry fines, could be obtained.

Aggregate plant studies.

The construction schedule provided a concreting period of approximately 18 months. On this basis it was estimated that the maximum concreting rate would be approximately 70,000 cubic yards per month, and the average rate about 56,000 cubic yards. With these estimates as a basis, the following fundamental design assumptions in respect to aggregates were made:

Capacity: 300 tons per hour, average.

Separation: Coarse aggregate, 4 size gradations; fine aggregate (sand), 2 size gradations.

Gradation retained on screen size:	*Percent*
6-inch	0
3-inch	23
1½-inch	17
¾-inch	12
⅜-inch	8
No. 4 mesh	5
No. 8	7
No. 14	7
No. 28	7
No. 48	5
No. 100	5
Passing No. 100 mesh	4
	100

Storage: 30,000 tons (3 days' supply at peak production).

Originally two schemes for processing aggregates were proposed. As far as coarse aggregate production was concerned, the two schemes were identical. The essential difference lay in the manner of

producing sand or fine aggregate. Scheme 1 was based on dry separation throughout the plant, with a division of the sand at the No. 14 size and the removal of excess fine material from the No. 14 mesh material by air separation or screening. Scheme 2 contemplated washing the sand to remove the excess fines, but did not provide for separation of sand into two size fractions. After consideration of the two schemes, taking into account the conditions under which the plant would be operated, a third scheme was suggested which in effect was identical with scheme 1 except that washing was substituted for dry removal of excess fines.

The third scheme offered the greatest possibilities and with some modification was finally adopted. Considerations upon which this decision was made were based primarily upon the desire for a high-quality concrete. This could be obtained in scheme 3 because of the greater plant flexibility and closer fine aggregate control, even though it necessitated additional plant facilities and additional storage space. Although dry separation of fine sand would eliminate the moisture-control problem, it had the disadvantages of increasing the dust hazard, of probably being impossible because of the damp nature of weather conditions in the area, and of causing undesirable segregation which would partially defeat the purpose of separation into two sizes.

When the problem of general plant design had been settled, the problem of procurement presented itself. Two methods of procuring the desired plant were outstanding: First, a complete plant could be purchased completely installed and ready for operation. Second, each item of the plant designed by the Authority could be purchased separately and assembled into a complete plant by the Authority's construction forces. In addition, each scheme presented the further possibility of buying either used or new equipment. As a matter of policy this alternative was settled favoring the purchasing of new equipment since the dependability of a plant composed of used equipment would be questionable.

After considerable study of the two schemes of procurement, weighing the advantages and disadvantages of each, a decision was reached to invite bids: First, for a plant of the Authority's design both for parts of the plant and for the entire plant installed and ready for operation; and second, for a plant of the manufacturer's design installed and ready for operation, the Authority merely giving the general lay-out and the tonnage rate and gradation required. There were advantages favoring both the purchase of the separate items of the plant and of the purchase of the plant completely installed and ready for operation. This being the case, it was decided that the price consideration and dependability should govern the final decision as to the method of procurement.

Five bids were received for a complete plant furnished and erected by the bidder. These bids ranged from $346,800 to $236,450. After making the adjustments necessary to bring the bids to a comparable basis, which involved eliminating the proposed sand plant and eliminating bids which were too high for consideration, the bids ranged from a high of $268,992 to a low of $260,467. In a study of these bids after the comparisons were made, the high bid of $268,992 was

selected as the best value for a complete plant erected in place and ready to operate. However, it was estimated that on the basis of the Authority's purchasing the separate items and equipment and erecting the plant, the cost of these same facilities would be reduced to $229,000. This plant was comparable in every way to the plant offered by the acceptable bidder except that storage facilities would be reduced from 40,000 tons to 30,000 tons. The difference of $40,000 in price was estimated to be considerably more than necessary to provide the additional storage. This comparison of costs, when combined with the advantages favoring buying separate items and erecting the plant by the Authority's construction forces, was the deciding factor in determining the method of procurement to be used.

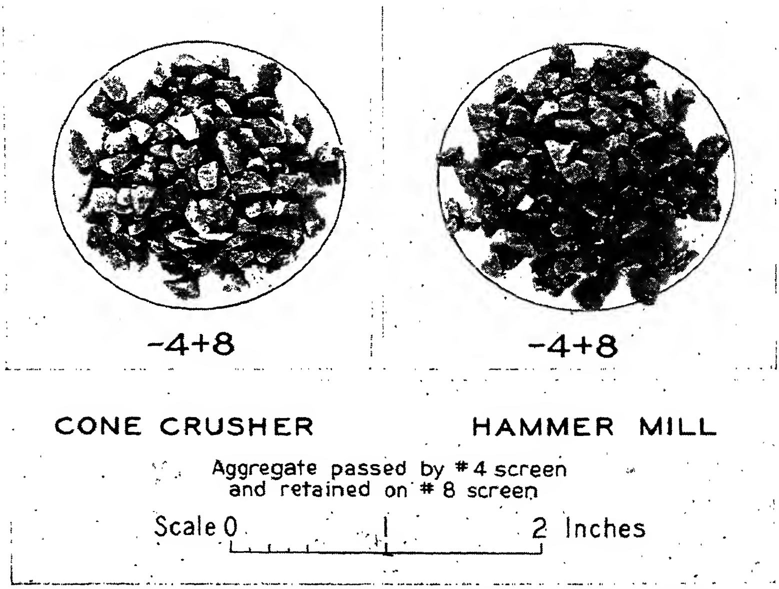


FIGURE 111.—*Comparison of hammer mill and cone crusher products.*

The design and selection of the sand plant involved considerable inquiries as to the type of crushers, arrangement and type of screens and conveyors, and type of wet classifier that would produce the desired product. It was evident from the beginning that particle shape, gradation, and the ability to control gradation would be the factors which would have the greatest effect on the quality of concrete produced and cement required. A program of investigation was inaugurated to supplement the work that had already been done, in which a number of plants were observed to determine what particle shape and gradation could be expected from several types of crushers under consideration when crushing dolomite. In addition, samples of dolomite were sent to several manufacturers of crushing equipment

who crushed the material in the type of crusher which they proposed to furnish. Samples were obtained from four hammer mills, a gyratory crusher, a cone crusher, a roll crusher, and a rod mill. Samples of the products from each of these mills were obtained for studies of particle shapes and general characteristics. A more complete statement of these tests is given in appendix E. These tests produced two striking conclusions: First, the shape of the gradation curve from all the machines was very similar; and second, the particle shapes from the several types of machines varied greatly, with the rolls producing an elongated, or splinter-shaped particle, and the hammer mills producing a product approaching a cube in shape, with products of the other machines somewhere between the two. The first of these conclusions indicated very definitely that selective screening and washing to get rid of some of the minus 100-mesh material would be necessary. The second of these conclusions resulted in the elimination from further consideration of all the machines except the hammer mills and cone crushers. From further testing these two types, the hammer mills were selected as producing the best fine aggregate. The two principal reasons for the selection of hammer mills were that a cement saving of from 4 to 5 percent was indicated if the hammer mills were used, and that the difference in initial cost of the two types of machines favored the use of hammer mills.

The aggregate manufacturing plant, for purposes of administration and discussion, has been divided into, first, the crushing and screening plant consisting of crushers and screens necessary to produce coarse aggregate (material retained on a 3/8-inch square mesh screen); and, second, the sand plant consisting of hammer mills, screens, and washers necessary for producing fine aggregate or sand.

Coarse aggregate manufacturing plant.

Quarried rock was delivered to the primary crusher where it was crushed to pass a 6 5/8-inch opening—the maximum opening of the crusher. A small surge bin beneath the crusher received the crushed material and by means of a vibrating feeder distributed it on a 36-inch conveyor belt that carried the rock to a scalping screen above the secondary crusher. This screen had an upper deck with 7 1/2-inch openings and a lower one with 3 1/2-inch openings. All material which was retained on the top deck was passed through the secondary crusher along with any desired proportion of that retained on the lower deck; the remainder of the rock was passed by stone ladders to the 30-inch conveyor belt leading to the screening structure.

A magnet located near the end of the first conveyor removed any tramp iron that came from the quarry or primary crusher to prevent damage to the cone crusher. Later on, this was supplemented by two fixed electro-magnets at the sand transfer bins.

Material from the secondary crusher was carried to the screening structure by the same belt which carried the material that bypassed this crusher. When this material reached the screening structure, it was picked up by a short belt that delivered it to the 3-inch opening cobble screen. Rock retained on this screen was dropped to a storage pile and that which passed went by belt to a pair of 1 7/8-inch screens operating in parallel. Any portion of the material retained on this

pair of screens or the two pairs following could be diverted to the hammer mill feed collecting conveyor. The next screens in line were 7/8-inch mesh or medium rock screens, and they were followed by the fine rock screens with an opening of 3/8 inch.

The structural steel screening structure was built above a reinforced concrete tunnel. Gates were provided in the roof for reclaiming each of four sizes of rock and two sizes of sand. All stored rock was dropped to the storage piles from their respective screens. Varying needs for different sizes of stone necessitated control of the amount of each size produced. This control was exercised for cobbles by varying the amount sent around the secondary crusher from the scalping screen and for the other three sizes by the amount taken for hammer mill feed. Figure 113 shows the equipment used in the plant and the percentage of the total aggregate handled by each unit.

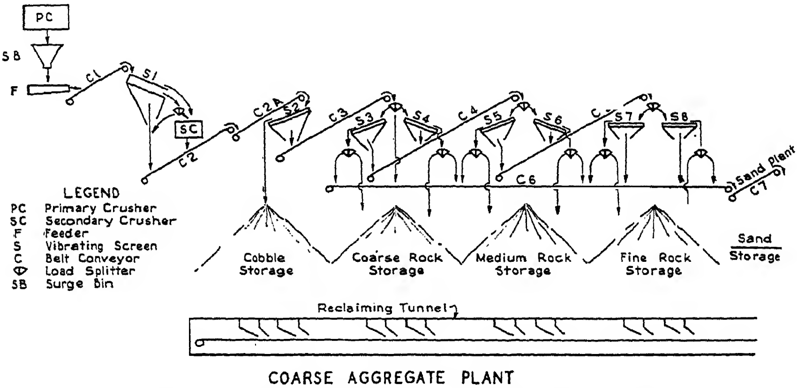


FIGURE 112.—*Flow of coarse aggregate through the plant.*

This plant, ready to run, originally cost \$156,125.06. Table 58 summarizes the first cost of major units and their installation cost.

TABLE 58.—*First cost and installation cost of major parts of the coarse aggregate plant*

Equipment cost:[1]		
Crushers	\$47, 834. 78	
Conveyors	26, 550. 67	
Screens	9, 177. 88	
Feeders	1, 311. 92	
Stone ladder, chutes, and signal system	1, 688. 65	
Steel screening structure	12, 816. 98	
		\$99, 380. 88
Installation cost:[1]		
Primary crusher	20, 905. 66	
Secondary crusher	8, 860. 01	
Conveyor and screens	26, 978. 51	
		56, 744. 18
Total equipment and installation costs		156, 125. 06

[1] Costs are for equipment as installed as of February 1, 1935, without additions or betterments installed at later dates.

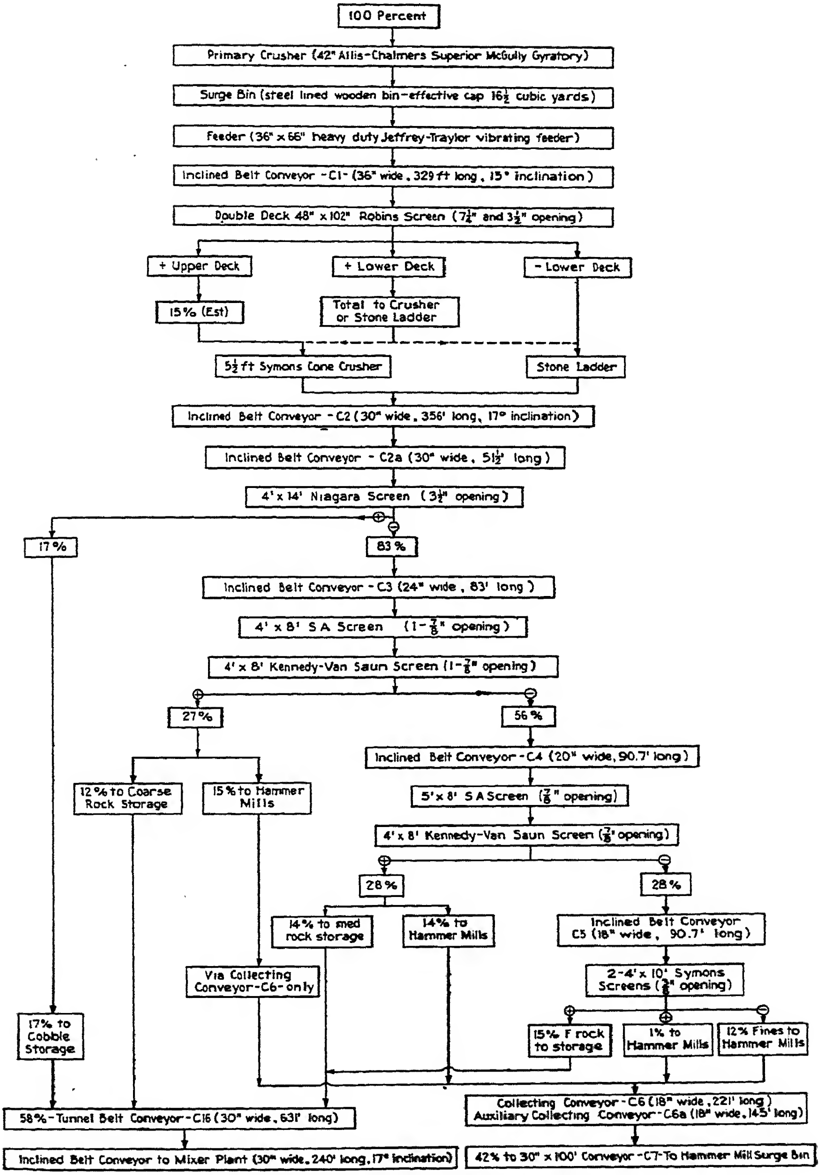


FIGURE 113.—*Coarse aggregate flow sheet.*

The personnel and rates of pay for a normal crew needed to operate this plant are given below:

Number	Classification	Rate per hour
1	Foreman	$1.00
1	Hoist operator at primary crusher	1.00
1	Carpenter (1 shift per day)	1.00
3	Mechanics' helpers	.75
4-5	Laborers	.45

Performance.—All of the aggregate used in more than 1,000,000 cubic yards of concrete and for certain other miscellaneous work, such as rock for roads, was produced by this plant. Production began in June 1934 and continued into the early part of 1936. The total amounts of aggregate produced and the approximate percentage of each are shown in table 59.

TABLE 59.—*Total aggregate produced*

	Tons	Percent each size in concrete	Percent of rock produced
Cobbles	349, 639	30. 8	16. 8
Coarse rock	231, 106	20. 4	11. 1
Medium rock	268, 082	23. 6	12. 9
Fine rock	286, 626	25. 2	13. 8
Total (dam concrete)	1, 135, 453	100. 0	54. 6
Roads and other facilities	62, 843		3. 0
Sand plant (dam concrete)	873, 363		42. 0
Waste (dirty fine rock)	8, 113		. 4
Total rock crushed	2, 079, 772		100. 0

Plant production averaged 287 tons of rock per net operating hour. Only approximately 7½ percent of the shift time was lost due to shut-down and 4 percent lost because of intermittent feeding that caused the equipment to operate with no load. During February 1935, the peak month, the plant averaged 350 tons per loaded hour; and this may be considered as the capacity of the plant for sustained operation. The primary crusher, however, was rated at 615 tons per hour.

As previously mentioned, the coarse aggregate was carried to four piles over a reclaiming tunnel. The estimated amount of each size in storage under average conditions was:

Material	Percent of gross storage	Average gross storage	Live storage
		Tons	*Tons*
Fine rock	12	7, 800	1, 760
Medium rock	30	19, 500	4, 850
Coarse rock	35	22, 800	5, 690
Cobbles	23	15, 000	3, 700
Total	100	65, 100	16, 000

Due to inequalities of production, the storage piles were not built up uniformly, and the fine rock storage became the limiting factor. The amount of material shown in the preceding tabulation as live storage, except in the case of the fine aggregate, could be increased by pushing the rock into the crater caused by withdrawal. The fine rock segregated and the material on the side of the pile was not suitable for use in concrete. A 2-day break-down in June 1935 provided a good check on the live storage, and 1,757 tons of fine rock were used before the available supply was exhausted.

Cost of operation.—The cost of quarrying rock and delivering it to the crusher was $0.374 per ton. These costs were divided into five divisions as shown in figure 110.

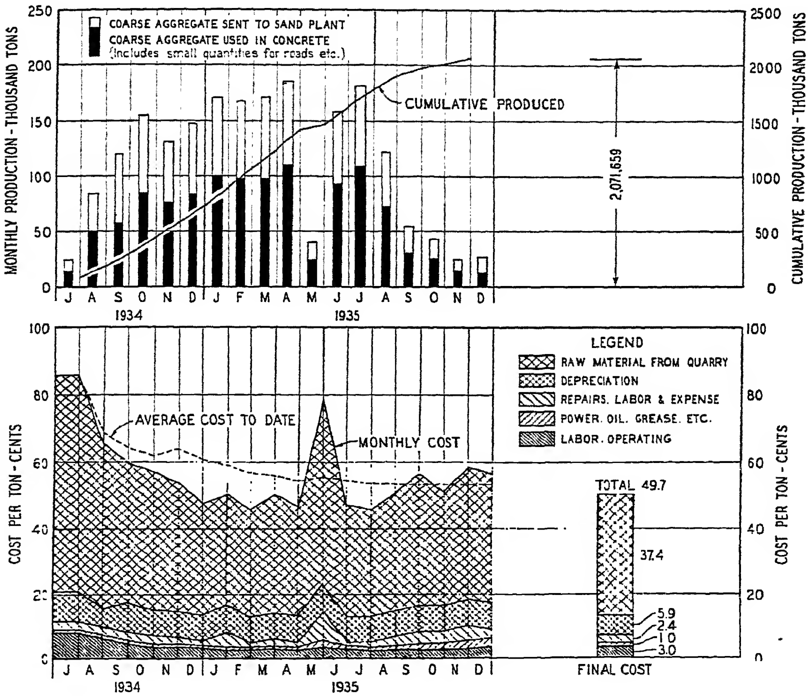


Figure 114.—*Coarse aggregate production—Quantities and costs.*

To crush, screen, and deliver the stone to storage or to the hammer mills cost $0.123 per ton, giving a total cost of stored rock of $0.497 per ton. A special study showed that practically 20 percent of the cost of crushing and screening the rock was repair cost; 25 percent, operating labor; 6 percent, power, oil, and grease; and 49 percent, depreciation of plant equipment. Details of crushing and screening costs based on 2,071,659 tons are shown in table 60.

TABLE 60.—*Analysis of crushing and screening costs*

	Total cost	Cost per ton	Total cost per ton
Operation:			
Labor	$61, 913. 18	$0. 030	
Power, oil, grease, waste rock	[1] 19, 447. 82	. 009	
			$0. 039
Repairs:			
Primary crusher	15, 746. 91	. 008	
Secondary crusher	9, 888. 34	. 005	
Conveyors and chutes	12, 934. 00	. 006	
Screens	11, 938. 05	. 006	
			. 025
Depreciation:			
Crushing and screening plant	122, 755. 61		. 059
Total (crushing and screening plant)	254, 623. 91		. 123
Quarrying rock, less dirty fine rock (see fig. 110)	774, 874. 29		. 374
Total (coarse aggregate stored and delivered to hammer mills [2])	1, 029, 498. 20		. 497

[1] Includes $3,034.26 for 8,113 tons of fine rock which was too dirty for use in concrete. This money was deducted from total cost of rock from quarry.
[2] Total cost of 873,363 tons of rock delivered to hammer mills is $432,161.01.

Changes in installation.—Probably the most general change required throughout the plant was in the size of the screen cloth opening. In general, the openings in the screens originally installed were too small to make the separations of rock desired. The material very near to the size of the screen opening tended to pass over the screen. During the period when the quarry was being stripped, considerable clay was mixed in with the rock. This clay caused blinding of the lower deck fine-rock screens in wet weather. This continued intermittently until quarrying on the main floor was completed. The initial and final set-up of the screen cloth openings, but not the intermediate sizes used to arrive at the satisfactory arrangement, are shown below:

Screens	Size of rock desired, inches	Initial screen installation, inches	Final screen installation, inches
Scalping:			
Top deck	+6	6	7¼
Lower deck	+3	3	3½
Cobble	+3	3	3½
Coarse rock	+1½	1½	1⅞
Medium rock	+¾	¾	⅞
Fine rock	+4 mesh	¼	⅜

Almost at the beginning of the plant operations, it was apparent that one screen each would not handle the coarse rock or the medium rock without overloading and poor screening. On September 1, 1934, an additional screen was installed for each of these sizes.

The heavy loads on the Stephens-Adamson fine rock screens caused excessive breaking of the stabilizer rods. When the capacity of the sand plant was increased, these two screens were moved to the sand plant where the loads were lighter, and were replaced with two 4- by 10-foot Symons horizontal two-deck screens that proved satisfactory for this work.

When the production of mass concrete was at its height, a bypass placed between the cobble and coarse rock screens directed a portion

of the material to the conveyors of the hammer mill feed. This arrangement relieved the coarse and medium screens from having to screen material that was to be crushed into sand when the demand for their sizes was the greatest. Use of this bypass was continued for about 5 months.

The vibrating feeder below the primary crusher was efficient but required frequent replacement of plates. These 5/16-inch plates were replaced with 3/8-inch ones once, but the vibration was so dampened that this size plate could not be used.

A ratchet and pawl stop was designed for use on the 36-inch conveyor from the primary to the secondary crusher to prevent the conveyor from running backward if it had to be stopped while loaded. The pawl missed several notches and then caught, stopping the belt so suddenly that the impact bent the head pulley shaft and damaged the magnetic pulley so badly it was removed and replaced with a steel-head pulley and a lifting magnet. The new shaft which was put in was bent when a similar slipping occurred. A job-built hold-back arrangement for checking backslips was then installed and the ratchet and pawl discarded.

Several conveyor changes were made to take care of the increased capacity of the sand plant when additional hammer mills were installed.

Repairs.—In addition to the repairs and changes made in the device to prevent backslipping of the conveyor belt to secondary crusher, there was only one major repair to the coarse aggregate crushing and screening plant equipment.

In January 1935 the primary crusher choked, and as a result, a ring on the spider that served to hold in position the thrust ring at the top of the main shaft was broken. This permitted the shaft to lift upward and allow dust and rock chips to drop down into the eccentric and so damage the babbitt that it was necessary to replace the eccentric.

A machined ring was welded in the spider, and the tap bolts were extended into the old casting. Two days were required for the repair. A steel spider was ordered for the crusher, but no further trouble developed and it was never used. The eccentric that was removed was rebabbitted and placed in stock.

The carbon steel diaphragm wearing plates had to be replaced often and were finally replaced with manganese steel plates. This reduced the frequency of the change. No accurate account was kept of how much work was done before a set of plates was worn out, but the operating forces estimate that the manganese steel lasted about twice as long as the carbon steel. Oil was changed in the primary crusher every 4 or 5 months, and the oil pump was overhauled once.

Numerous spare parts were held in stock for this crusher, as this piece of equipment was a vital part of the crushing plant.

No parts were replaced on the secondary crusher. It was dismantled once for cleaning and inspecting and was adjusted from time to time as the size of the feed was changed. Oil was changed about every 5 months, and the oil pump was removed once for cleaning.

Because of the heavy work done by the scalping screens, replacement of screen cloth was frequent during the first part of produc-

tion but was reduced by the use of skid bars over the screening. Parts causing most repairs were the transverse springs and hook-bolt outfit.

All chutes were made on the job and in most cases required frequent relining; where it was possible, stone boxes were built instead of the straight type chute to lessen wear.

All minor servicing of the belt was done by the operating crews, and no belt was replaced although several were worn badly.

Sand plant.

Sand plant feed from the coarse aggregate screening plant, for the first two hammer mills installed, was received in surge bins equipped with magnetic feeders, which maintained constant supply

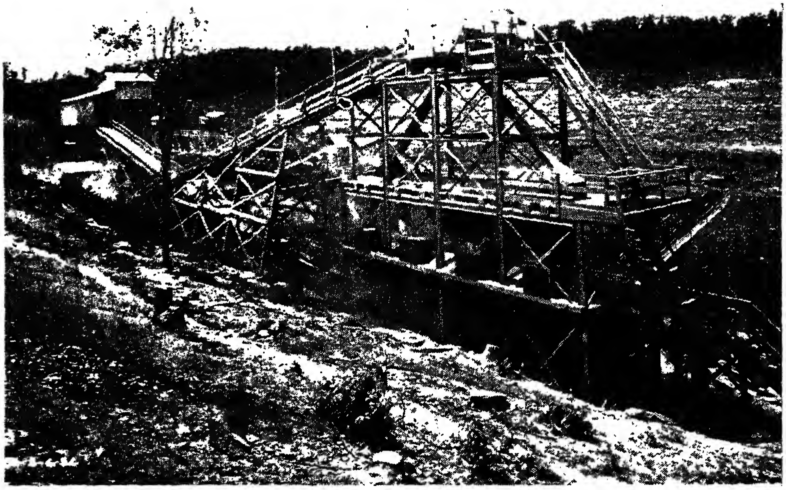


FIGURE 115.—*Sand plant.*

to the hammer mills. The additional two hammer mills which were installed some time later were fed by belt conveyor. A lifting magnet was placed at the feeders to remove stray particles of metal. Surge bin capacity was sufficient to care for all normal fluctuations and feed.

Hammer mill installation consisted of four units. Two of these were 42- by 48-inch Allis-Chalmers 4A pulverizers operating at 900 revolutions per minute, and were included in the original plant installation. Two additional hammer mills, 42- by 47-inch Pennsylvania SRX 100 Ajax, also operating at 900 revolutions per minute, were later installed for increased sand production and to provide a spare mill for emergency use. Two of the four were originally equipped with slugger-type hammers and had adjustable lower breaker plate and adjustable grate bars spaced at 2 inches. Feed was introduced from the top along the vertical center line of the mill. The other two units were equipped with stirrup hammers and

had fixed breaker plates and fixed grate bars spaced at 1¼ inches. Feed was introduced on the up-running side of the mill through a gravity chute in a line intersecting the center line of mill and at about a 45° slope. Effective inside dimensions were essentially the same for both mills; that is, 4 feet long by 42-inch diameter hammer circle. Each mill was direct connected to a 250-horsepower, 880-revolution-per-minute slip-ring induction motor. During the last few months of operations, stirrup-type hammers were used in both types of mills.

The hammer mills were operated in a closed circuit with six vibrating screens. All of the oversize from top deck (plus ¼-inch) was returned to the hammer mills for further crushing. Oversize from lower deck (plus 8-mesh) or coarse sand could be either returned to hammer mills together with plus ¼-inch material or

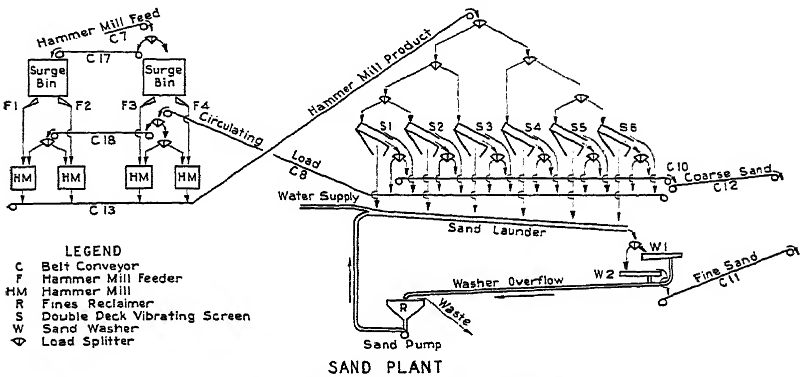


FIGURE 116.—*Flow of fine aggregate through plant.*

sent to coarse-sand storage. Normally, it was estimated that about 40 percent was returned to the mills. Circulating loads varied from a minimum of about 25 percent to a maximum of about 100 percent (based on net output), and averaged about 50 percent. The circulating load passed a 1-inch square hole. Variations in circulating load were due to variations in the condition of hammers, breaker plates, grate bars, and to some extent to variations in screening efficiency. Since screening was done dry, 8-mesh separation was considerably affected by moisture during rainy weather. The minus 8-mesh material went to washers, where the excess fines were removed.

Initial sand production was low and metal wear high, and a large amount of experimental work was necessary to develop the desirable operating characteristics later found possible with this equipment. The effect of type and number of hammers, grate bar spacing, and other factors governing sand production was studied in detail. A summary of these studies is given in appendix E. Final results were very satisfactory, and there was no doubt that the use of the hammer mills for making sand had been justified from either the point of quality or of cost.

Four of the six double-deck vibrating screens in the sand plant were 4- by 10-foot Allis-Chalmers "Aero-Vibe" motor-driven vibrating screens, and two were 47- by 86-inch Jeffrey-Traylor magnetic vibrating screens. Since cleaner screening was obtained by the magnetic screens, all of the coarse sand from these screens was

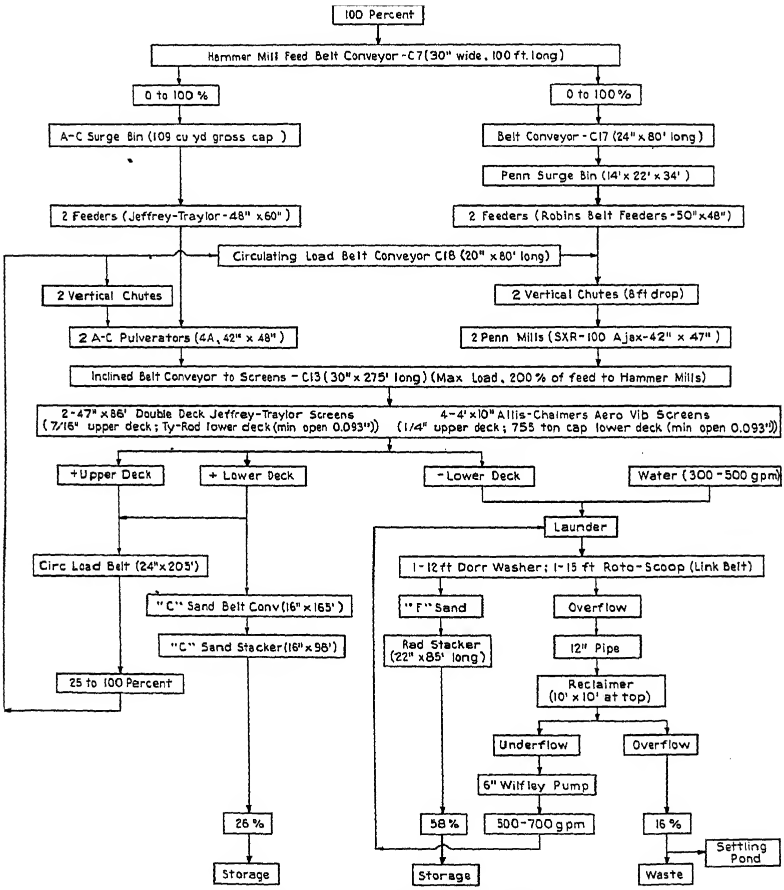


FIGURE 117.—*Fine aggregate flow.*

stock-piled for use in concrete; and as much as possible of the coarse sand from the motor-driven screens was returned to the hammer mills. Screen cloth with approximately 0.09-inch opening was used on all lower decks. "Ton cap" cloth with the long opening at right angles to material flow was used on the motor-driven screen, and "ty-rod" cloth with the long opening parallel to material flow was used on the magnetic screen. Hammer-mill product could be

screened satisfactorily by such slotted cloth since the percentage of thin elongated particles was small. Three-eighths- and seven-sixteenths-inch square opening cloth was used on the top decks of the magnetic screens; and ¼- or $\frac{5}{16}$-inch square opening cloth was used on top decks of the motor-driven screens. A ½-inch square opening would have been desirable for all top deck screens, but screening capacity of the lower decks limited opening sizes of top decks. The deficiency in 4-mesh to ⅜-inch size, which may be noted on the combined aggregate grading curves (fig. 182), was due to the compromise in cloth size between fine rock screens (Symons) and the top decks of the sand plant screens.

FIGURE 118.—*Sand washing plant.*

Two sand washers, one 12-foot Dorrco and one 15-foot Link-Belt Rotoscoop, washed and dewatered the fine sand, removing excess fine rock and the small amounts of clay which were present. The original installation included only the Rotoscoop, but unsatisfactory initial operation and low capacity of this machine made the addition of a second washer imperative.

An excess of water was required to transport washer feed from the screens to the washers; and as a result, the final product was often overwashed, and screen analyses of the finished fine sand were erratic. In order to control definitely the "fines" (minus 100-mesh material), a "fines" reclaimer was installed. This apparatus, constructed on the job, was essentially a wooden tank built in the shape of an inverted pyramid with overflow weirs along the four edges and an outlet at the apex. Overflow from the sand washer was received in the center of the reclaiming tank, which was baffled to decrease turbulence. A Wilfley sand pump took water and solids from the apex of the pyramid (bottom of tank) and discharged them into the trough carrying washer feed to the sand washers. In opera-

tion, the larger particles settled more rapidly and made their way to the sand pump suction while the light particles were overflowed with the waste water. Adjustments in quantities of water used in overflow levels and weir lengths controlled within close limits the amount of 100-mesh material retained in the fine sand without appreciably increasing the quantity of extremely fine particles (minus 325-mesh).

During the latter part of the job, waste water from the reclaimer was piped to a settling pond where reclaiming fines were settled out for use in foundation grouting and as agricultural limestone.

First cost of the major units of this sand plant and the installation costs as determined by a special analysis are summarized in table 61.

TABLE 61.—*First cost and installation cost of major parts of the sand plant*

Equipment Cost:		
Hammer mills and housing	$44, 630. 51	
Conveyors	12, 766. 63	
Stackers	3, 309. 15	
Screens	4, 639. 21	
Classifiers	6, 619. 07	
Settling tank	1, 923. 69	
Lifting magnets	2, 463. 31	
Signal equipment	79. 90	
Miscellaneous	422. 96	
		$76, 854. 43
Installation Cost:[1]		
Hammer mills and housing	25, 915. 45	
Conveyors, screens, structures	20, 035. 80	
Sand classifiers and settling tank	10, 048. 26	
General expense	586. 67	
		56, 586. 18
Total equipment and installation cost		$133, 440. 61

[1] These costs do not include additions and betterments after operation.

The normal operation crew for the plant and the rate of pay per hour for each classification were:

Number	Classification	Rate of pay per hour	Total cost per hour
1	Foreman	$1. 00	$1. 00
1	Mechanic	1. 00	1. 00
1	Carpenter	1. 00	1. 00
1	Electrician	1. 00	1. 00
2	Mechanic's helpers	. 75	1. 50
1	Pump operator	. 75	. 75
1	Oiler	. 60	. 60
4	Laborers	. 45	1. 80
	Total for crew		8. 65

A mechanic, a carpenter, and two laborers, costing $2.90 per hour, were charged to repairs; and the remaining members of the crew, costing $5.75 per hour, were charged to operations.

Performance.—The total gross feed to the hammer mills during the period between July 1934 and January 1936, the period of maximum operation, was 864,714 tons, while the sand produced in this period was 732,430 tons, showing a loss of 15.3 percent of hammer-mill feed. The

major portion of this loss was "fines" washed away in the overflow water, of which 23,000 tons were reclaimed. The average for the period was 121 tons per net operating hour. In March 1935 the highest hourly production rate of 147 tons per net operating hour was reached. The rate for March was about the maximum amount that could be produced by this plant. Net operating time was about 6.5 percent less than the total shift or gross hours. The sand plant was

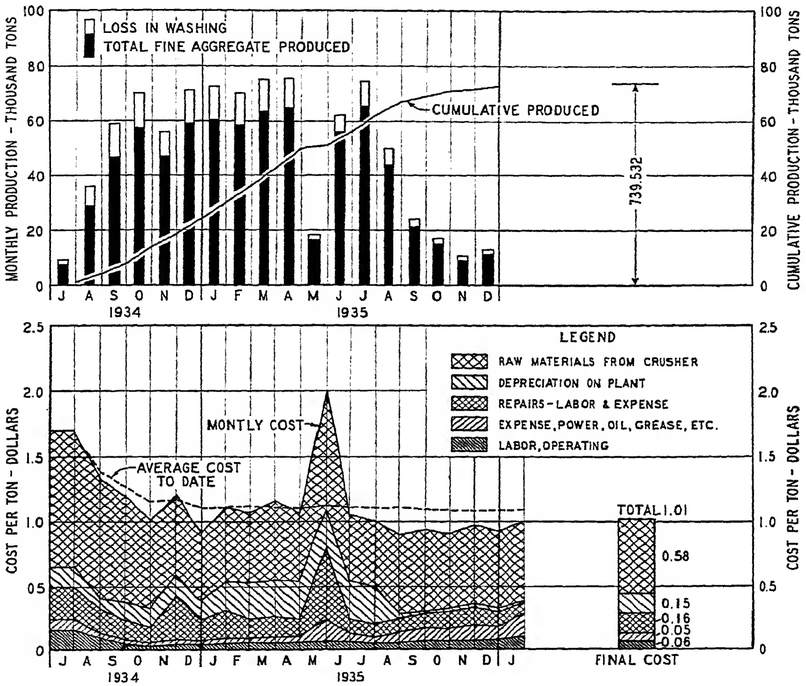


FIGURE 119.—*Sand plant operation—Quantities and costs.*

operated intermittently after January 1936 until May 1, 1936. During this period 7,594 tons of sand were produced, and 1,055 tons of fines were removed by the washer.

Samples for screen analyses were taken for control at least once a day, and average screen analyses representing at least 30 samples were calculated at the end of every month. All hammers were accurately weighed before and after being used. Tonnages were prorated to the various mills and in turn to hammers, liners, and grate bars on a power-consumption basis. The average load on each motor was calculated from recording ammeter charts for every 24-hour period, and horsepower-hours obtained by multiplying by the net operating hours. Recording ammeter charts also served to keep an exact record of net operating time of each mill and the net operating time that each set of hammers or other wearing parts remained in the mills. At the end

of every month, the tonnage produced was estimated from tonnages going through the weighing batchers at the mixer plant. An adjustment was made for either a gain or loss in storage. Since under normal conditions this gain or loss constituted a small percentage of the total production, any error that may have been made in estimating would be negligible when applied on the total tonnage for the month. The error on periods longer than 1 month would be smaller. From the tonnage figure and the total horsepower-hours for the month, the tonnage per horsepower-hour was calculated. This factor was used in distributing the tonnage to the mills and to the various kinds of hammers used.

An 85-foot long radial stacker, with a 22-inch belt, distributed the washed fine sand in a long storage pile over a section of the reinforced concrete tunnel through which sand was reclaimed. The distribution of the fine sand over the long storage pile permitted it to drain, so the materials delivered to the mixing plant had a relatively low moisture content. In order to get sand with low moisture content, removal of sand storage was usually done as far away from the stacker as possible. This resulted in the live storage of fine sand being about half full. The approximate sand storage under average conditions was:

	Coarse	Fine
Total storage capacity	7,800	24,000
Possible live storage	2,700	8,000
Amount in live storage	1,700	4,000

Lowest draw-down of coarse sand occurred when repairs were made to the head pulley of conveyor belt No. 1; at this time 1,440 tons were used without exhausting the live storage.

Cost of operation.—Unit cost of sand produced is based on an amount of 740,024 tons (includes 501 tons in unused concrete) from 873,363 tons of rock sent to the hammer mills. Hammer mill feed costs $0.497 per ton; but when computed on the basis of sand produced, becomes $0.584 per ton because 15.3 percent of the gross feed to the mills was washed out by the sand classifiers. An analysis of the fine aggregate costs is given in table 62.

Table 62.—*Analysis of fine aggregate costs*

	Total cost	Cost per ton	Total cost per ton
Operation:			
Labor	$46,224.09	$0.063	
Expense (power, oil, etc.)	38,016.03	.051	
			$0.114
Repairs and replacements:			
Hammer mills [1]	76,512.95	.103	
Classifiers	7,991.57	.011	
Conveyors	18,463.22	.025	
Screens	14,675.39	.020	
			.159
Depreciation	113,623.88		.154
Hammer mill feed:			
873,363 tons @ $0.497	432,161.01		.584
Total cost	747,668.14		1.011

[1] Includes repair costs to chutes and liners.

Installation of the reclaimer previously described provided a saving in cost of concrete in three ways:

1. Percentage of sand produced in relation to hammer mill feed was increased (saving $9,948).
2. Amount of sand produced in relation to total aggregate used in concrete mixers was reduced (saving $8,573).
3. Cement content per cubic yard was cut (saving $23,921). Savings shown here are for a period from April 1, 1935, to December 31, 1935.

The reclaimer was made on the job, and no exact cost was available. The cost used below is an estimate by job forces:

Cost of reclaimer (estimate)	$5,000
Power	216
Pump operator	2,025
Miscellaneous inspection, etc	100
Total—Installation of operations	7,341
Gross savings	42,442
Net savings	35,101

Changes in installation.—A number of changes were made in the plant and several experiments in the manner of operation were made. The majority of the changes were made at the time of and as a result of the installation of the second two hammer mills.

Clay from the quarry caused excessive blindings of the screen in the sand plant during wet weather. In an attempt to remedy this, wet screening was tried for 3 days in February 1935. Perforated pipes with hoods over them were installed to spray water upon the upper decks of the screen. The lower decks received water from the upper ones. This manner of screening was not satisfactory and was abandoned because:

1. The excess of water necessary to operate the screens provided so much water for the sand washers that overwashing resulted.
2. Wet circulating load with the wet materials from the fine rock screens caused excessive wear in the hammer mills.
3. Additional water in coarse sand and fine rock made it very difficult to control water-cement ratio for the concrete.
4. Mud and water freezing up in conveyors made their operation difficult.

To increase the capacity of the 15-foot Link-Belt Rotoscoop washer a new rotor with three blades instead of six and of a slightly smaller diameter was installed. Even with these changes the resulting capacity of 75 tons per hour was not sufficient to care for job needs.

To remedy this deficiency, a 12-foot Dorrco sand washer was installed. The machine was free from repair troubles, but because of the turbulence of the washer and the smaller settling area too many of the fines were washed from the sand. A slow-speed socket that reduced the rotor speed from 2 to 1⅔ revolutions per minute was installed to cut down the turbulence, and an auxiliary discharge trough was welded on so the discharge weir could be moved back and the settling area increased.

Material passing the lower 8-mesh deck of the fine-rock screen during 5 months of production was piped directly to the washers through

a bypass from the coarse aggregate screening structure. This relatively coarse material placed in the washers tended to accentuate the coarseness of the sand, and its use was discontinued after that trial period.

To care for the load on the screens when the crushing capacity was increased, two screens were added to the four already installed. The two Stephens-Adamson 5- by 8-foot vibrating screens that were originally installed as the fine-rock screens were moved to the sand plant. They were rebuilt from single to double deck screens but never gave satisfactory service because of broken bearings, sheared oil pipes, and stabilizer breakage. No satisfactory method of fastening the screen cloth was provided when the screens were rebuilt and excessive wear resulted in high cost for this item. After about 5 months of service these screens were discarded, and Jeffrey-Traylor magnetic vibrating screens were installed in their place. These proved to be the most satisfactory screens used for the sand plant because of their low maintenance cost and because of their more efficient separation of both dry and moist material. Beyond a certain amount of moisture, however, these screens would blind as badly as the other makes.

The reclaimer was installed to improve the quality of sand and reduce waste. Original installation provided a centrifugal pump that soon wore out because of the abrasiveness of the particles and the resultant wear upon the impeller. When a 6-inch Wilfley sand pump was installed, operation of the unit was satisfactory.

Experiments indicated the possibility of using rock flour for foundation grouting. For this use and its use as agricultural lime, a pipe was installed from the end of the reclaimer discharge pipe to the upper end of the ravine between the primary and secondary crushers. Here a bulkhead was built to form a settling basin for the very fine material carried by this waste water. From June until November 1935 about 23,000 tons of material were reclaimed at a cost of $8,657.

Repairs.—The sand plant was remarkably free from recurrent and expensive repairs. Except for the Stephens-Adamson screens that were finally discarded, very little screen repair work was necessary. One bearing was replaced on one of the Allis-Chalmers screens.

No repairs were necessary on the Link-Belt washer, and the only repair necessary on the Dorrco machine was made to a broken cast steel spider—the result of attempting to start the machine under a load. The break was welded and a steel plate was bolted under the center of the rotating spider. This type of repair proved so satisfactory that a new spider was never installed.

With the exception of the hammer mill replacements and, perhaps, chute repairs, the most frequently recurring replacements were screen cloth. With the introduction of a different type cloth, and taking full advantage of the facilities for tightening the cloth used, this cost was cut considerably. The production of each screen cloth usually varied between 30,000 and 10,000 tons. Normally each screen handled one-sixth of the hammer mill products delivered to the screening plant, an average of 31 tons per hour.

Control and signal equipment.

All of the main parts of the aggregate manufacturing plant were controlled by manually operated equipment. All motors driving screens and conveyors of the coarse and fine aggregate screening plants were controlled by fusible magnetic switches from start-stop push-button transfer switch stations arranged for either group or individual operation.

The signal equipment consisted of lights and howlers which were actuated by a system of relays interlocked with the control equipment of the plant. The signal system power was supplied through 440/110-volt auto-transformers which received power from the same source as did the motors and controls.

The electrical equipment was divided into four groups for purposes of control and signalling. The divisions were made at points in the plant chosen so that the groups became separate units with definite operating relationships, both within the unit and with adjacent units. Group 1 included the primary crusher and the feeder from the primary crusher surge bin. Group 2 included the conveyor from primary to secondary crusher, the scalping screen, and the secondary crusher. Group 3 included the conveyor from the secondary crusher to the rock screening structure, all of the equipment in the rock screening plant, and the conveyor from the rock screening plant to the surge bin serving the first two hammer mills. Group 4 included all of the sand plant.

The control and signal system was arranged to operate for certain definite conditions which were anticipated as likely to occur in the starting, running, and stopping of the plant. Certain other conditions which were also likely to occur were not provided for in the control and signal system, but were the responsibility of the operators. Fundamentally, the control and signal system was arranged to fulfill the following conditions:

1. Signal all parts immediately affected by the operation of any other part of the plant.
2. Prevent starting of certain related parts except in proper sequence.
3. Afford flexibility of operation to certain contingencies which were bound to arise.

As far as signal and control are concerned, the sand plant (group 4) was entirely separate from the large aggregate plant (groups 1, 2, and 3).

Aggregate data.

Average screen analyses of the aggregate produced by both the coarse and fine aggregate plants are given in table 63. Tables 64 to 69 inclusive show average monthly screen analyses for the six aggregate sizes. Fineness of the minus 100-mesh material contained in the finished fine sand is shown in table 70. Fineness of waste material washed out of the fine sand is included for comparison. An inspection will show that there is no definite separation at any particle size, but that the washing greatly reduces the percentage of extremely fine particles remaining in the fine sand. Average chemical analyses of the dolomite aggregate computed from monthly analyses of composite fine sand samples are shown in table 71. Specific gravity of all aggregate sizes was 2.82 for nearly all samples tested at the dam.

TABLE 63.—*Average screen analyses of sand plant products and concrete aggregate*

[Percent passing]

Screen size	Sand-plant products week ending Jan. 26, 1936: Hammer mill feed	Hammer mill products	Circulating load	Coarse sand	Fine sand	Concrete aggregates in use as of Jan. 29, 1935: Cobbles	Coarse rock	Medium rock	Fine rock	Coarse sand	Fine sand
6-inch	------	------	------	------	------	100.0	------	------	------	------	------
5-inch	------	------	------	------	------	85.5	------	------	------	------	------
4-inch	------	------	------	------	------	------	------	------	------	------	------
3-inch	------	------	------	------	------	12.9	100.0	100.0	------	------	------
2½-inch	100.0	------	------	------	------	------	------	------	------	------	------
2-inch	67.5	------	------	------	------	------	------	------	------	------	------
1½-inch	64.3	------	------	------	------	2.5	6.8	88.8	100.0	------	------
1¼-inch	54.5	------	------	------	------	------	------	------	------	------	------
1-inch	43.8	100.0	100.0	------	------	------	------	------	------	------	------
¾-inch	28.4	96.5	97.0	------	------	1.2	1.1	8.8	95.2	------	------
½-inch	22.3	93.2	90.5	------	------	------	------	------	------	------	------
⅜-inch	17.7	87.2	77.5	------	------	.9	.7	1.5	38.1	------	------
3-mesh	14.1	78.1	56.0	100.0	100.0	.8	.6	1.3	14.1	100.0	100.0
4-mesh	11.4	68.4	37.9	86.0	99.9	.7	.6	1.0	4.2	82.6	99.9
8-mesh	7.1	50.4	14.3	23.4	98.7	.5	.5	.8	2.2	22.6	97.0
14-mesh	------	37.3	7.1	4.0	69.2	.4	.4	.8	1.6	6.3	66.7
28-mesh	------	29.1	5.6	3.1	42.4	.3	.4	.7	1.4	4.5	42.0
48-mesh	------	21.2	4.8	2.9	24.8	.2	.3	.6	1.2	3.8	27.0
100-mesh	------	17.7	3.9	2.7	11.5	.2	.2	.5	1.1	3.2	14.9
Number samples	5	5	5	2	2	3	6	4	4	10	18

TABLE 64.—*Average screen analyses of concrete aggregate cobbles* [1]

	Percent passing: 6-inch	5-inch	4-inch	3-inch	2½-inch	2-inch	1½-inch	1¼-inch	1-inch	¾-inch	½-inch	⅜-inch	3-mesh	4-mesh	8-mesh	14-mesh	28-mesh	48-mesh	100-mesh
1934																			
August	100.0	94.9	70.2	38.2	18.3	8.2	5.1	4.4	3.6	2.6	2.1	1.8	1.5	1.3	1.0	0.9	0.7	0.6	0.5
September	100.0	94.2	65.5	22.7	10.2	3.7	2.7	2.4	2.0	1.6	1.3	1.1	.9	.8	.6	.4	.4	.3	.3
October	100.0	97.3	75.7	37.0	14.6	6.8	4.4	3.7	3.0	2.3	1.9	1.6	1.4	1.2	.9	.7	.6	.5	.4
November	99.1	86.3	60.4	16.8	8.1	5.2	3.5	2.9	2.4	1.7	1.4	1.2	1.0	.9	.7	.5	.4	.3	.2
December	100.0	85.8	60.3	20.0	10.1	6.4	3.8	3.1	2.6	1.8	1.5	1.3	1.1	1.0	.8	.6	.5	.4	.3
1935																			
January	100.0	89.3	55.2	16.2	8.6	5.5	3.8	3.4	2.9	2.2	1.9	1.6	1.3	1.2	.9	.6	.5	.4	.3
February	100.0	91.0	63.9	22.4	10.4	5.6	3.4	2.8	2.3	1.7	1.3	1.1	.9	.7	.6	.5	.4	.3	.2
March	100.0	86.9	55.3	17.0	8.8	5.2	3.8	3.2	2.7	2.0	1.6	1.3	1.0	.8	.6	.4	.3	.2	.2
April	98.4	87.6	61.0	23.0	10.9	5.8	3.9	3.1	2.6	1.8	1.5	1.2	1.0	.9	.7	.5	.4	.3	.2
May	100.0	91.8	59.6	18.8	7.8	3.4	2.0	1.7	1.4	1.0	.9	.7	.6	.5	.4	.3	.2	.2	.1
June	98.8	87.5	56.9	21.6	11.6	6.4	4.5	3.8	3.4	2.6	2.1	1.8	1.5	1.2	.9	.7	.6	.4	.3
July	99.3	87.3	61.0	20.1	9.2	4.9	3.1	2.5	2.1	1.5	1.2	1.0	.8	.7	.5	.4	.3	.2	.2
August	98.0	88.0	59.0	19.9	11.1	5.9	3.8	3.1	2.7	2.0	1.6	1.4	1.1	1.0	.7	.6	.5	.4	.3
September	99.5	89.8	62.1	17.1	9.8	6.2	3.8	3.1	2.7	1.9	1.5	1.2	.9	.8	.6	.4	.3	.3	.2
October	99.0	86.9	59.7	19.3	11.0	6.8	4.2	3.4	2.9	2.1	1.5	1.2	.9	.8	.6	.4	.3	.3	.2
November	98.3	89.4	55.8	19.0	12.3	8.6	5.8	4.9	4.2	3.3	2.6	2.2	1.8	1.5	1.1	.9	.7	.6	.4

[1] Cloth size: August–Sept. 20, 1934, 6-inch upper, 3-inch lower; Sept. 20, 1934–March 1935, 6½-inch upper, 3½-inch lower; April–August 1935, 7¼-inch upper, 3½-inch lower. Approximate number of analyses per month: 1934, 6; 1935, 12.

TABLE 65.—*Average screen analyses of concrete aggregate coarse rock* [1]

	Percent passing																
	4-inch	3-inch	2½-inch	2-inch	1½-inch	1¼-inch	1-inch	¾-inch	½-inch	⅜-inch	3-mesh	4-mesh	8-mesh	14-mesh	28-mesh	48-mesh	100-mesh
1934																	
August	------	100.0	92.1	65.0	32.4	16.9	6.7	3.0	1.7	----	----	----	----	----	----	----	----
September	------	100.0	92.1	66.5	29.2	13.0	5.8	1.6	1.0	----	----	----	----	----	----	----	----
September [2]	100.0	98.6	81.8	48.4	20.8	9.6	4.2	1.6	1.1	0.9	0.8	0.7	0.6	0.5	0.4	0.4	0.3
October	100.0	------	71.8	40.2	12.6	5.7	2.0	.7	.6	.5	.4	.4	.3	.3	.3	.2	.2
November	100.0	------	77.6	43.6	13.3	6.3	3.0	1.7	1.4	1.2	1.0	.8	.8	.7	.6	.6	.5
December	100.0	------	71.1	30.7	3.5	.9	.4	.1	.1	.1	.1	.1	.1	.1	.1	.1	.1
1935																	
January	100.0	------	74.2	37.7	7.8	2.9	1.8	1.2	.9	.8	.6	.6	.5	.4	.3	.3	.2
February	100.0	------	75.7	36.2	5.9	2.0	1.1	.8	.6	.5	.5	.4	.3	.3	.2	.2	.1
March	100.0	------	79.0	42.3	8.4	3.2	2.0	1.4	1.0	.8	.7	.6	.5	.5	.4	.3	.2
April	100.0	97.6	79.0	45.0	9.3	3.3	2.0	1.5	1.0	.9	.7	.7	.6	.5	.4	.4	.3
May	100.0	97.8	78.3	39.9	6.7	2.4	1.5	1.2	.9	.9	.8	.7	.6	.5	.4	.4	.3
June	100.0	96.2	79.9	46.9	10.5	3.4	1.7	1.3	.9	.8	.7	.6	.5	.5	.4	.4	.3
July	100.0	96.3	78.8	48.3	13.7	4.5	2.2	1.8	1.4	1.0	.7	.6	.6	.5	.5	.4	.3
August	100.0	97.5	83.8	50.3	12.5	4.8	3.1	2.2	1.4	1.2	1.0	.8	.7	.6	.5	.4	.4
September	100.0	93.4	76.7	43.9	9.0	4.1	2.5	1.8	1.2	1.0	.8	.7	.6	.6	.5	.4	.3
October	100.0	91.0	72.9	44.5	11.2	4.1	2.1	1.4	.8	.7	.7	.6	.5	.4	.4	.3	.3
November	100.0	89.0	75.3	45.6	11.7	3.6	2.3	1.7	1.2	1.1	1.0	.9	.8	.7	.6	.5	.5

[1] 3-inch screen not used in making analyses of coarse rock, October 1934–March 1935. Cloth size: August–Sept. 20, 1934, 3-inch upper, 1½ inch lower.
[2] Sept. 20, 1934–August 1935, 3½-inch upper, 1⅞-inch lower. Approximate number of analyses per month: 1934, 12; 1935, 24.

TABLE 66.—*Average screen analyses of concrete aggregate medium rock*

	Percent passing													
	2-inch	1½-inch	1¼-inch	1-inch	¾-inch	½-inch	⅜-inch	3-mesh	4-mesh	8-mesh	14-mesh	28-mesh	48-mesh	100-mesh
1934														
August	------	100.0	91.3	74.2	27.5	7.1	3.6	------	------	------	------	------	------	------
September	------	100.0	94.3	76.5	28.8	7.1	2.1	1.2	------	------	------	------	------	------
September [1]	------	100.0	86.3	67.1	23.4	6.9	3.0	1.9	1.3	1.1	0.9	0.8	0.7	0.6
October	100.0	96.1	84.8	63.9	16.3	3.6	1.4	1.0	.9	.8	.7	.6	.5	.5
November	100.0	------	77.2	56.2	9.4	2.6	1.3	1.1	1.0	.8	.7	.6	.6	.5
December	100.0	84.4	64.2	40.1	6.7	.8	.3	.3	.3	.3	.3	.2	.2	.2
1935														
January	100.0	84.8	64.9	42.5	8.5	2.2	1.4	1.1	.9	.8	.7	.6	.5	.4
February	100.0	86.6	63.5	40.0	7.0	1.9	1.1	.9	.8	.7	.6	.5	.4	.4
March	100.0	89.1	67.2	45.1	9.0	2.6	1.7	1.3	1.1	1.0	.9	.7	.6	.5
April	100.0	87.7	65.2	42.8	6.9	1.8	1.1	.8	.7	.6	.6	.5	.4	.4
May	100.0	85.1	61.8	37.3	5.6	1.9	1.4	1.3	1.2	1.1	1.0	.9	.8	.6
June	100.0	89.8	68.7	44.6	7.7	2.1	1.4	1.0	.9	.8	.8	.7	.6	.5
July	100.0	89.3	68.3	45.9	7.8	2.3	1.5	1.2	1.1	1.0	.9	.8	.7	.6
August	100.0	91.9	71.1	48.0	9.2	1.9	1.2	.9	.8	.7	.6	.6	.5	.5
September	100.0	91.3	69.8	46.7	8.7	1.7	1.1	.8	.7	.6	.6	.5	.5	.4
October	100.0	90.4	64.9	41.5	7.9	1.4	1.0	.9	.8	.7	.6	.6	.5	.4
November	100.0	89.8	66.7	41.6	9.0	1.7	1.2	1.0	.9	.8	.7	.6	.6	.5

[1] Sept. 20, 1934–August 1935, 1⅞-inch upper, ⅞-inch lower. Approximate number of analyses per month: 1934, 12; 1935, 24.

NOTE.—1½-inch screen not used in making analyses of medium rock during November 1934. Cloth size: August–Sept. 20, 1934, 1½-inch upper, ¾-inch lower.

TABLE 67.—*Average screen analyses of concrete aggregate fine rock*

	Percent passing										
	1-inch	3/4-inch	1/2-inch	3/8-inch	3-mesh	4-mesh	8-mesh	14-mesh	28-mesh	48-mesh	100-mesh
1934											
August	100.0	99.5	89.2	58.1	28.3	22.7	3.9	3.1	2.6	2.2	1.7
September	100.0	98.9	84.2	48.8	16.1	3.2	1.7	1.3	1.2	1.0	.8
October	100.0	96.0	71.3	36.9	15.3	7.0	3.9	2.9	2.4	2.1	1.8
November	100.0	92.5	56.3	17.1	2.5	1.4	1.1	1.0	1.0	.9	.8
December	100.0	93.5	67.9	35.5	13.7	5.5	2.7	2.4	2.1	1.7	1.4
1935											
January	100.0	96.0	73.4	46.1	25.4	14.5	8.1	5.8	4.6	3.8	2.8
February	100.0	94.6	69.8	43.0	21.2	10.8	5.7	4.1	3.4	2.8	2.2
February [1]	100.0	94.6	66.3	32.4	9.0	4.3	2.7	2.4	2.1	1.8	1.5
March	100.0	94.5	66.2	33.6	10.2	5.5	3.8	3.2	2.7	2.4	2.0
April	100.0	92.1	61.0	29.0	6.6	3.2	2.3	2.0	1.7	1.5	1.2
May	100.0	90.2	59.9	27.0	7.2	4.1	3.1	2.8	2.4	2.1	1.8
June	100.0	92.5	62.0	26.3	5.0	2.3	1.7	1.6	1.4	1.3	1.1
July	100.0	92.1	55.0	22.8	4.9	2.7	1.9	1.7	1.5	1.3	1.1
August	100.0	93.4	56.6	22.1	3.2	1.6	1.1	1.0	.9	.8	.7
September	100.0	95.7	60.3	23.0	3.8	1.7	1.0	.9	.8	.7	.6
October	100.0	94.4	51.5	17.5	4.0	2.7	2.2	2.0	1.8	1.5	1.2
November	100.0	95.6	56.1	19.4	2.5	1.2	1.0	.9	.8	.7	.7

[1] Feb. 11–August 1935; 7/8-inch upper, 3/8-inch lower. Approximate number of analyses per month: 1934 12; 1935, 24.

NOTE.—Cloth size: August–Sept. 20, 1934; 3/4-inch upper, 1/4-inch lower. Sept. 20, 1934–Feb. 11, 1935, 7/8-inch upper, 1/4-inch lower.

TABLE 68.—*Average screen analyses of concrete aggregate coarse sand*

	Percent passing								
	1/2-inch	3/8-inch	3-mesh	4-mesh	8-mesh	14-mesh	28-mesh	48-mesh	100-mesh
1934									
August			100.0	94.9	13.2	2.4	1.8	1.6	1.4
September			100.0	83.2	18.2	5.5	4.2	3.5	2.7
October			100.0	79.6	14.8	4.2	3.2	2.7	2.2
November			100.0	85.4	22.4	4.6	3.4	3.1	2.6
November [1]		100.0	94.6	79.4	33.4	14.4	10.9	9.0	7.4
December			100.0	76.9	13.2	2.6	1.6	1.3	1.2
1935									
January			100.0	82.9	20.8	5.3		3.1	2.6
February		--	100.0	76.0	18.2	6.3	3.6	3.0	2.5
February [2]		100.0	92.0	64.1	14.9	5.7	4.5	3.7	2.9
March		100.0	94.4	73.1	21.9	8.4	6.3	5.3	4.3
April		100.0	96.3	78.5	23.9	7.4	5.2	4.3	3.4
May		100.0	95.4	77.6	20.2	5.4	4.0	3.3	2.7
June		100.0	92.9	69.2	14.2	3.3	2.8	2.2	1.7
July	100.0	99.8	93.0	72.5	15.3	5.9	4.9	4.0	3.1
August	100.0	99.9	92.8	71.9	15.8	5.7	4.2	3.6	3.0
September	100.0	99.9	89.2	67.0	8.9	2.3	1.8	1.6	1.4
October	100.0	99.6	89.6	63.0	9.3	2.5	1.9	1.8	1.6
November	100.0	99.6	86.5	57.3	6.9	2.4	2.2	2.0	1.9

[1] Nov. 18–Dec. 2, 1934, 3/8-inch upper, 8-mesh lower. Dec. 2, 1934–Feb. 17, 1935, 1/4-inch upper, 8-mesh lower.

[2] Feb. 17–August 1935, 1/4-inch on 4 screens, 3/8-inch and 7/16-inch on 2 screens upper, 8-mesh lower.

NOTE.—Cloth size: August–Nov. 18, 1934, 1/4-inch upper, 8-mesh lower. Approximate number of analyses per month: 1934, 25; 1935, 30.

TABLE 69.—*Average screen analyses of concrete aggregate fine sand*

	Percent passing						
	3-mesh	4-mesh	8-mesh	14-mesh	28-mesh	48-mesh	100-mesh
1934							
August	100.0	99.9	95.5	61.3	35.4	20.2	11.6
September	100.0	99.7	96.2	64.6	39.2	23.3	13.1
October	100.0	99.8	98.2	66.1	39.6	24.2	13.5
November	100.0	99.9	98.2	67.5	41.8	25.6	14.0
December	100.0	99.8	97.7	69.8	42.1	25.5	12.9
1935							
January	100.0	99.8	97.4	66.4	40.9	25.5	13.5
February	100.0	99.8	98.1	68.0	42.3	25.4	13.0
March 1–25	100.0	99.8	96.8	65.9	40.4	24.3	12.0
March 25–31	100.0	99.9	98.2	68.5	43.3	27.6	16.0
April	100.0	99.9	97.7	69.2	43.4	27.2	15.8
May	100.0	99.9	97.4	69.0	43.8	28.1	16.9
June	100.0	99.9	97.4	68.9	44.0	28.9	18.0
July	100.0	99.9	96.5	75.1	42.9	27.6	17.1
August	100.0	99.9	95.7	67.7	42.9	27.4	17.3
September	100.0	99.9	93.7	64.2	40.4	25.7	16.7
October	100.0	99.9	92.9	61.2	37.9	23.4	15.0
November	100.0	99.9	92.2	60.0	36.2	22.6	14.0

NOTE.—Cloth size: 755 T. C., 341 T. C., Tyrod, and others used with no appreciable effect on grading. Sand reclaimer installed Dec. 27, 1935. Wilfley sand pump installed at reclaimer Mar. 25, 1935. Approximate number of analyses per month: 75 (each sample a shift composite).

TABLE 70.—*Fineness analyses*

[Percent finer than particle size indicated]

Particle size microns	—100-mesh material in washed fine sand	—100-mesh material in reclaimer waste
(74) 200-mesh [1]	41	91
(44) 325-mesh [1]	27	58
40	21	40
35	18	36
30	16	32
25	14	28
20	12	23
15	10	18
10	8	14
5	5	9
1	1	4

[1] Sizes in parentheses determined by wet screening, other sizes determined by Bouyoucos hydrometer.

TABLE 71.—*Chemical analysis of dolomite concrete aggregate*

Components:	*Percent*
Silicon dioxide (SiO_2)	5.6
Ferric oxide (Fe_2O_3)	.4
Aluminum oxide (Al_2O_3)	1.4
Calcium oxide (CaO)	29.2
Magnesium oxide (MgO)	19.8
Ignition loss and miscellaneous	43.6

Analysis is average of monthly analyses of composite sand samples.

CEMENT HANDLING

A total of 1,107,676 barrels of bulk cement was used in construction work. The maximum daily consumption anticipated was 3,630 barrels. Actual maximum monthly consumption was 94,280 barrels for April 1935. Two days' supply, or about 6,000 barrels, storage capacity at the dam was established as a safe minimum.

Three methods of hauling the cement from the mill to the job were studied. These three methods were, briefly, (1) to haul the cement by truck direct from the Volunteer Portland Cement Co. mill at Knoxville, a distance of 33 miles by the shortest route; (2) to haul the cement from this mill by the shortest existing paved road, a distance of 45 miles; and (3) to haul the cement by rail to Coal Creek and then by truck to the dam. The first and second methods were estimated to effect a small saving in transportation cost. These were rejected, however, because approximately one-fifth of the total cement used would have to be transported by rail before the roads could be made ready for use, because the long truck haul increased the possibility of accidents by 12 times and involved moral responsibility for damage to the highways due to heavy hauling, and because shipments would be limited to a single cement mill.

The plan adopted provided that the cement be shipped by rail to Coal Creek where unloading facilities, a storage silo, and truck-filling facilities were provided. The bulk cement was then hauled 4.6 miles by four special semi-trailer tank trucks to the dumping hopper and storage silo at the dam. Two stationary pumps conveyed the cement by pipe lines from the hopper to the mixing plant bin, or the storage silo. Cement was reclaimed from the silo by a chute which returned the cement by gravity to the same hopper where it was pumped to the mixer plant bin. Box cars, bulkheaded back of the doors and lined to prevent loss of the cement in transit, were specified for shipping the cement.

Unloading facilities at Coal Creek.

Since railroads did not have hopper-bottom cement cars available for use in this locality, methods for unloading the cement from box cars had to be developed. Two plans were studied: the first scheme consisted of the use of portable cement pumps which picked up cement from the boxcar floor and conveyed it by air to the top of the silo; the second scheme used a manually-guided power scraper pulled by a winch to scrape cement into a hopper. From the hopper a screw conveyor carried the cement to a vertical bucket elevator which dumped into the top of the silo. Although the first cost of the second scheme was somewhat less than that of the first scheme, the first scheme was chosen. This greatly reduced the physical hardship to the operator caused by cement dust which is much greater with the scraper method. The scheme chosen was more adaptable and flexible for possible use on other projects. Two Fuller-Kinyon portable pumps were purchased for this work.

The storage silo was a structural steel tank with a conical bottom, having a level full capacity of 6,000 barrels. The average amount stored in the silos was less than 2,500 barrels. This was kept low to provide a large empty space above the cement to allow the cement to settle, resulting in little loss of aerated cement dust through the

vent. Jets of compressed air at the discharge opening of the silo were used to keep the cement from clogging the discharge valve. The overlapping joints of the steel plates of the silo were painted with asphalt to keep out moisture.

The two portable Fuller-Kinyon cement pumps were 6-inch, type B, No. 184, and had a rated capacity of 175 barrels per hour. Approximately 3 hours were required to pump out one car by use of a single pump. Approximately 1 hour and 50 minutes of this time were actually used in pumping. Thirty minutes were used in pulling bulkheads from the car; 20 minutes in moving the car in and out; 10 minutes as a rest period for the operator, because of the dust; and 10 minutes for oiling the machine. Using 1 hour and 50 minutes to pump out a car gave an average pumping rate of 129 barrels per hour. It was possible, however, to reduce this time to 1½ hours,

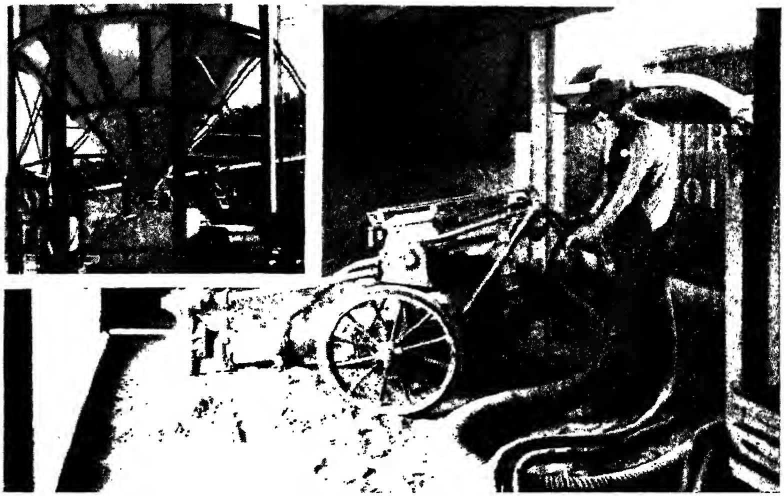


FIGURE 120.—*Cement unloading facilities at Coal Creek.*

which would be an average pumping rate of 160 barrels per hour. This higher rate was obtained by crowding the machine, which in time causes undue wear of the "Lenhart Seals." This seal seemed to be the machine's weakest point.

During the operation of these pumps several changes were made in their design because of the relatively high repair costs for certain parts. To reduce the wear on the hose and to make the hose easier to handle, small four-wheel carriages were installed. Much trouble resulted from foreign matter in the cement. Repair costs from this item were cut considerably by replacing a rivet on the collector plate which was supposed to shear if the plate hit any foreign material with a shear-bolt. Replacing the bolt took considerably less time than replacing the rivet.

The maximum output for a 6-hour shift as determined by special studies for the two pumps was six cars, or 1,440 barrels. This pro-

duction was obtained by extreme crowding of both machines and men. In July 1935, 384 cars were unloaded, or about 92,160 barrels. Actual working hours were 653½. Including the time the machines were out of order and other delays, the average production rate per machine was 70 barrels per hour, or 3 hours 26 minutes per car.

Hauling.

Four truck-tractors and tank-trailers were used for hauling cement to the dam. Each unit consisted of a model A–8 International tractor-type truck coupled to a 12½-ton model 817 Fruehauf semi-trailer equipped with a 65-barrel cylindrical tank. In order to conserve weight and permit a heavier pay load, the cement tanks were made of aluminum.

The extra first cost of four aluminum cement tanks over steel was $2,521. The aluminum tanks were built 5 feet in diameter and 16 feet long, against 5 by 13½ feet for a steel body. The saving in weight of 1,900 pounds allowed five barrels greater capacity for the aluminum bodies. The cost per load was the same and gross weight for both outfits was the same. The average load actually carried in the aluminum bodies was 57.9 barrels, and the estimated load for steel bodies was 52.9 barrels. This meant that 1,600 loads were saved by use of the aluminum bodies; including the extra cost of these aluminum bodies, approximately $2,000 was saved. Credit should also be given the aluminum bodies for no expense of painting, better appearance, and less depreciation.

The cement handling job could have been accomplished with three tank-truck units. The fourth was bought as an emergency unit and to enable the truck-tractor of the spare unit to be used for miscellaneous hauling work. There were two extra Fruehauf trailers—one a stake trailer and the other a pole type.

These trucks operated over the access road between Coal Creek and the dam. From Coal Creek to Norris the road first rises over a ridge, necessitating a climb of 486 feet in 2.6 miles. Thence the elevation drops 106 feet in 1.7 miles. The remaining 0.3 mile dropped 75 feet by gravel roadway to the cement silo.

When the trucks were being filled at Coal Creek the cement fell in a pile inside the tanks, leaving the tanks only partially filled. To help overcome this condition, when as much cement was loaded as possible, the trucks would turn around for the return trip and drive again beneath the silo for additional loading. This short run of approximately 200 feet vibrated the cement sufficiently to allow five or ten more barrels to be loaded. The average load of 57.9 barrels was approximately 89 percent of the manufacturer's rated capacity of 65 barrels.

The trucks were operated on the same shifts as was the remainder of the plant. Each unit was operated by a driver whose sole duty was to drive the unit, loading and dumping being taken care of by regular crews at the loading and unloading stations. As the trucks were operated, they could transport considerably more cement than was used. As a result, no cement shortage was experienced, and the trucks were occasionally out of service for repairs or for other work.

Four units operated a total of 15,106.5 hours to July 20, 1935, the principal period of activity. Eighty-nine percent of this time was

for hauling cement; 7.1 percent for hauling warehouse supplies; and 3.9 percent on miscellaneous hauling. The average time per load was approximately 40 minutes. The over-all average time per trip was 52 minutes.

The units were continuously subjected to the abrasive cement dust; therefore a daily greasing schedule was maintained, and the radiators were drained, flushed, and refilled frequently. Oil was changed every 500 miles.

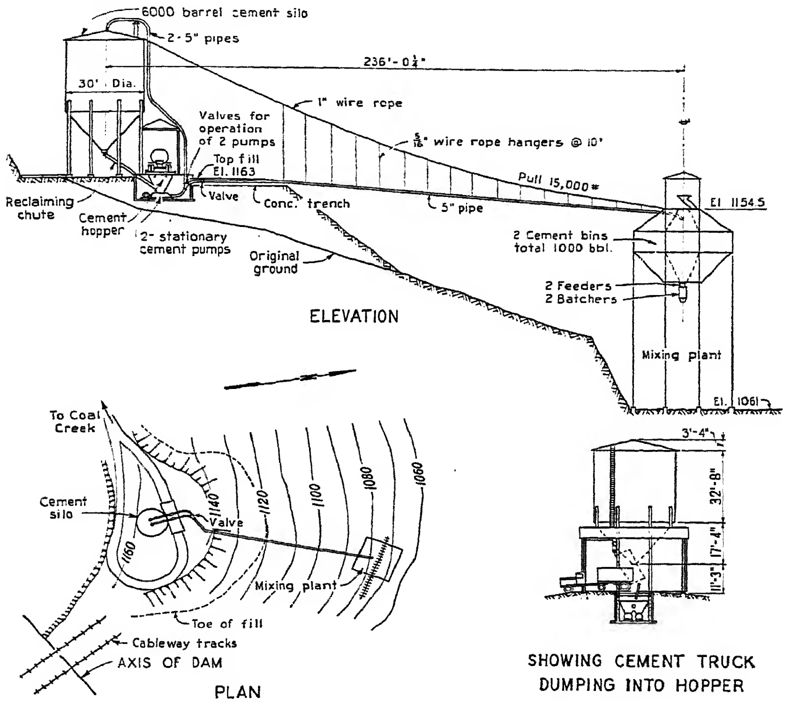


Figure 121.—*Cement handling facilities at the dam.*

The first cost of the four units delivered was $22,028. The total cost of operating the four units, including depreciation, was $49,099.91. For all uses to August 1, 1935, special studies showed the operating cost per hour to be approximately $3.10, or approximately $0.32 per mile. For hauling (1,107,676 barrels) the cost was $2,548 per load or $0.044 per barrel, or $0.048 per ton-mile.

Cement handling facilities at the dam.

Arrangements were made at the start of construction plant operations to provide space for a cement silo of 6,000-barrel capacity and dump truck facilities close to the mixing plant. Mechanical schemes as well as pneumatic were studied for conveying cement to the top of the mixing plant bin.

The silo was a duplicate of the silo at Coal Creek. A truck-dumping hopper and shed were located at one side of the silo. The dumping hopper was a structural steel cone under which two stationary Fuller-Kinyon cement pumps were placed in a concrete pit. The hopper gate was a steel section of the floor which the operator swung open after the truck had passed over it. An 8-ton electric monorail hoist hooked onto the front end of the tank lifted the front end, allowing the cement to flow out of the rear end into the hopper opening. Considerable dust was raised in the dumping operation, but this

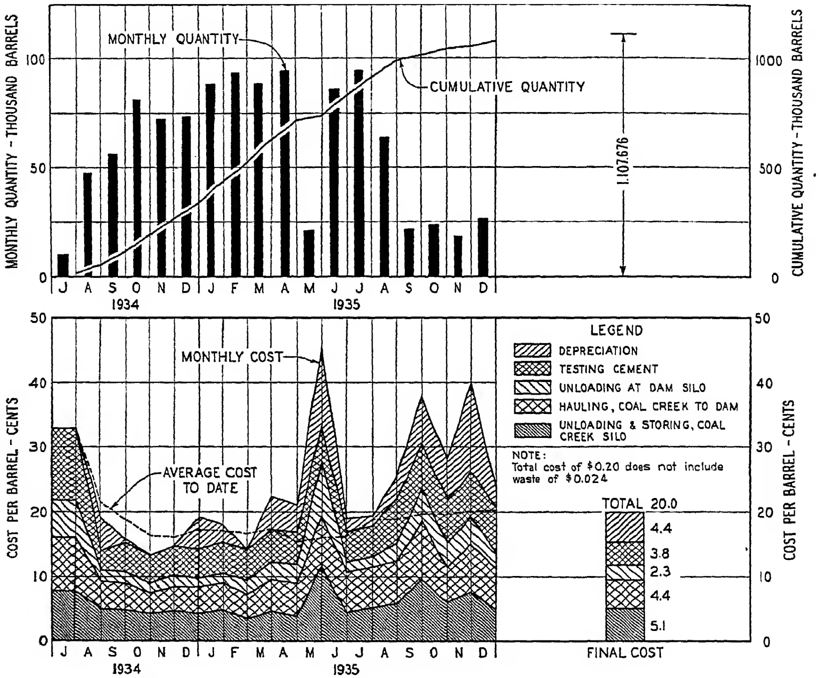


FIGURE 122.—*Cement handling—Quantities and costs.*

did not interfere with proper operation since the operator controlled the hoist dump at the front of the truck away from the dust.

Economical erection of the piping system and the flexibility of the lay-out, which allowed either pump to discharge to the silo or to the mixing bin, was the main advantage of this system over the others considered.

Cement loss.

Loss of cement in handling amounted to 1.37 percent of the total cement purchased, which covered the total loss including unloading at the box car, filling tank trucks, hauling, truck dump hopper, and loss of aerated cement through vents in silos and bins.

Cement handling costs.

The total original cost of cement handling facilities was $68,363.97. Repairs necessary because of settlement of the silos entailed an expenditure of $5,946.90 for new footings and replacement of columns and bracing at the dam silo, and an expenditure of $3,086.60 for new footings at the Coal Creek silo. As shown by a special study, the cost of handling 1,107,676 barrels of cement was $206,424.43, or $0.186 per barrel. If the cost of testing is included, the total cost of cement handling is increased to $249,523.07, or $0.224 per barrel.

CONCRETE MIXING PLANT

Prior to the selection of the mixing plant, extensive studies and investigations were carried on to determine a site that would tie in with the crushing and screening plant and the bulk cement storage

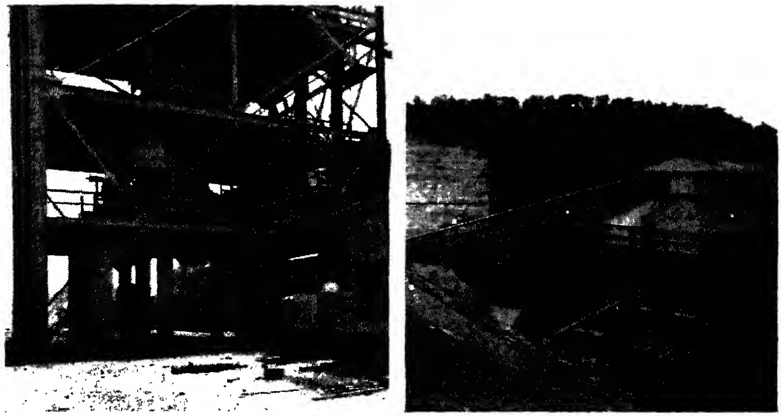


FIGURE 123.—*Mixing plant.*

system, and at the same time would be so located as to reduce the problem of concrete delivery to a minimum. This plant also had to be flexible to meet changes in all normal mixing and placing conditions, and at the same time maintain economy in production.

Some known conditions governing the design and construction of the plant were:

1. Aggregates were to come to the mixing plant by a conveyor system;
2. Two sizes of sand and four sizes of coarse aggregate were to be used;
3. Bulk cement was to be used, being pumped to the mixing plant from a cement silo, or dumping hopper;
4. Transfer trains and cableways were to be used in transferring and placing concrete.

Five general types of concrete mixes were to be used in the dam and powerhouse. Modifications of these general mixes and special mixes were to be used for conditions requiring concrete of a quality

not obtainable from the five standard mixes. Placing requirements called for the completion of the main portion of the dam between July 1934 and December 1935. This gave an estimated maximum monthly production of about 70,000 cubic yards. This rate of placing for a 26-day month and a 22-hour day meant a required average production of about 120 cubic yards per hour. Concrete dispatching schedule studies indicated that two cableways with 6-cubic-yard buckets and three 6-cubic-yard transfer trains properly scheduled under ideal working conditions could place a maximum of 180 cubic yards of concrete per hour. This was the basis for final selection of the plant.

OBJECT OF SIGNALS AND FUNCTION OF LEVERS	PERSONNEL	OPERATING INSTRUCTIONS
1 E [illegible] hopper 1 B [illegible] to start cement pumps 1 S [illegible] 2 B [illegible] regate feed gates [illegible] 2 blasts to stop. 2 S Horn acknowledging signal. 3 B Aggregate selecting signal buttons 3 S " " indicating light Light on, aggregate being sent. 4 B Button to start conveyors (interlocked)	1 HEAD HOUSE OPERATOR	A. Operate and maintain head house equipment B After sounding 1 blast on 2 B to tunnel operator and [illegible] C. [illegible] turnhead and repeat B D Determine contents of cement bins with sounding weight. [illegible] silo; receive 1 blast [illegible] signal. [illegible] with 1 B to silo. Receive 2 blasts acknowledgment, on 1 S. Cement stops.
5 B Toggle switch to set scales indicating desired mix. 5 S Signal light indicating type of mix in operation 6 B Toggle switch to cut out any batcher desired. 6 S Light indicating batcher which has been cut out	1 DISPATCHER	A Receive all instruction from dam by phone B. Set all batcher scales indicating type of mix desired, by 5 B C. Adjust water batcher. D Control movements of trains and cableways.
7 L Air levers for filling batchers. 7 B [illegible] batchers 8 B [illegible] 9 B [illegible] 10 B [illegible] 10 E [illegible] 11 E [illegible]	BATCHER OPERATOR NO 2	A [illegible] C [illegible] [illegible] full. D. Upon signal from operator No. 1 repeat operation
12 L Air levers for filling batchers. 13 L Air levers for dumping all aggregate, cement, & water. 13 S Light from mixer man to dump batchers. Light on = dump batchers. 13 B Switch to signal mixer man that batchers are empty. 13 E Air operated gates for dumping batchers. 14 S Blue light indicating water tank is being filled. 15 S Red " " " " " " discharged. 15 B Automatic water batcher control. 16 B Start button for motor driven turnhead.	BATCHER OPERATOR NO 1	A [illegible] 2 to fill batchers [illegible]
16 E Motor driven turnhead. 17 B Start and stop button for motor driven turnhead. [illegible] 17 E [illegible] turnhead and close signal circuit 5 13 17 L Air levers for dumping mixers mechanically interlocked with timers. 17 T Mix timer actuated by turnhead 16 E. Timer starts when turnhead leaves position after charging mixer. 18 E Mixer, air dumped. 18 B Signal to batch operator No. 1 to dump batchers (3 way sw) 18 S [illegible] 19 S [illegible] 20 B [illegible] 2 blasts proceed.	1 MIXERMAN	A. Check position of turnhead 16 E and if necessary adjust by means of 17 B. B. Signal batch operator with 18 B to dump batchers. C. [illegible] batchers are empty) repeat [illegible] D [illegible] release from mix timer and after seeing car in [illegible] E [illegible] train to move up. " 2 [illegible] " " " " proceed
20 E Concrete transfer car. 20 S Dispatching signal. 1 blast, move up; 2 blasts, proceed	TRAINMAN	A. After spotting train and receiving 1st batch, move ahead upon 1 blast from horn 20 S, wait for 2nd blast, then proceed.
	1 MECHANIC	A. Maintain and lubricate all equipment.
	1 CONC INSP	A. Control all conc proportions and consistencies. Do all testing.
	1 FOREMAN	A. Supervise, instruct operators and act as relief operator.

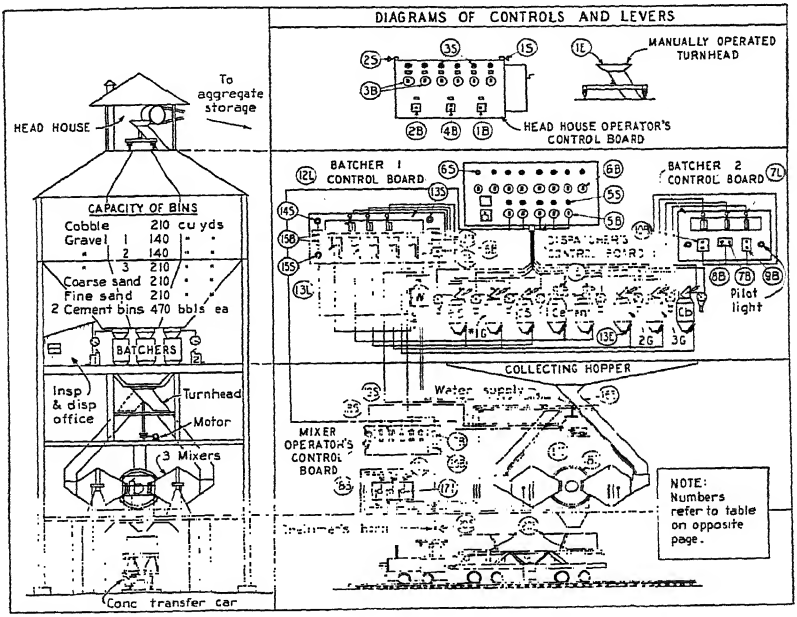
FIGURE 124.—*Schematic*

Description of plant.

The final equipment selected for handling and mixing the concrete made the plant one of the most modern and efficient of its kind. The plant consisted of three 3-cubic yard Smith, double-cone, tilting mixers designed for the maximum output of 180 cubic yards per hour, and all necessary equipment and controls for delivering material to the bins and for batching and mixing. The mixer building proper, a five-story steel structure, was 105 feet high. The building, steel-storage bins, and various other parts of the plant were erected with rivet bolts to permit easy dismantling and re-erection. The mixer support was a reinforced concrete structure separate from the rest of the building.

Aggregate for the concrete was reclaimed from the aggregate stock pile by a conveyor running through the reclaiming tunnel under the stock piles. Cement was brought to the mixing plant from the storage silo or from the truck dumping hopper. Raw water pumped from the Clinch River was used as mixing water.

The concrete aggregate received at the top of the mixing structure from the aggregate conveyor belt was directed to the proper bin by means of a manually operated turnhead. One bin was provided for each of the six aggregate sizes. Two bins were provided for cement. The aggregate bins were arranged symmetrically around the two cement bins which were located at the center of the structure. The



lay-out of mixing plant.

total bin capacity was about 1,100 cubic yards for aggregates and about 940 barrels for cement. One operator, aided by electrical signaling devices to sources of supply, kept the aggregate bin and cement bin filled.

All material flowed through the mixing plant by gravity. Located immediately below each storage bin was a C. S. Johnson air-operated batcher equipped with a Kron springless scale. All sand and aggregate filling gates were of the radial undercut type, with port openings of sufficient area to permit a gravity flow through them at a rate of 20 tons per minute when completely open. The discharge gate on each batcher hopper had a port opening of sufficient area to permit gravity discharge of the full contents in 7 seconds. All filling

and discharge valves were operated by air rams manually controlled and capable of practically instantaneous opening and closing. The rams were air cushioned at the end of the stroke to prevent damage when operated for instantaneous opening and closing of the batcher gates. The air valves for controlling the filling gates were adapted to group mountings and were so designed as to enable the operator to open or close instantly, to open or closely slowly, or to hold the filling gates in a partially open position. An automatic "jigging" device greatly aided in the accurate batching of cobbles, as it was impossible to add small increments of weight to the batch.

Each cement batcher hopper was equipped with an air ram operated, rotary-type filling gate, having a capacity of 3,000 pounds of loose cement per minute, and a motor-driven rotary vane-type feeder. This feeder was operated by a fully-enclosed gear system driven by a totally enclosed, fan-cooled, induction motor equipped with a quick-acting magnetic brake. At the top of the cement batcher unit was also an emergency sliding gate directly ahead of the filling gate. Discharge of the cement batcher unit was controlled by an air-operated gate faced with live rubber and covered with a dust guard. The opening controlled by each of these discharge gates was sufficiently large to permit the full batcher unit to be emptied in 7 seconds. Electric interlocks were provided to prevent operation of the filling gate when the discharge gate was open. Provision was made for the operation of the cement batcher filling gates by manual or automatic control, manual operation being preferred because of greater accuracy. Electrically driven vane agitators immediately below the cement bins, and air lines in the bins, assisted the cement to flow. The batcher discharge was assisted by a vibrator attached at the side of the cement batcher. A chute extended from the discharge of each cement batcher to the center of the charging hopper for preventing the cement from falling against the sides of the hopper and sticking, and for obtaining a better mingling of cement with the aggregates.

The scales with each batcher were of the springless type, swivel mounted, and adjustable for leveling. The dial mechanism was equipped with an adjustable dampening device for steadying the pointer movement. The scales were accurate to within 1 percent of correct weight.

Mixing water was batched automatically by a volumetric batcher of 200-gallon capacity. The measuring tank was enclosed and provided with an automatic overflow consisting of a motor-driven, screw-operated overflow riser within the tank. With this device, the amount of water per batch could be quickly changed. Gages showed the inspector and also the batcher operator the level in the tank. Water was delivered to the measuring tank through a 4-inch valve and discharged through a 5-inch valve. Both inlet and outlet valves were of the quick-acting type and were interlocked to prevent direct flow from the supply to the discharge line. Both valves were actuated by an air cylinder which, in turn, was controlled by an air valve located on the batcher operator control stand.

The batching of the material for each mix was divided between two operators, each being provided with a one-man control stand located on opposite sides of the batcher floor. Cement and three

aggregates were batched by one operator, and water and the three other aggregates by the second operator. The second operator dumped the batchers. Scale dials were in plain view of the operator; and pointers, carrying small electric lights, and attached to the dial faces, indicated the amounts of each material to be weighed.

Batched materials were dumped into a collecting hopper from which they were directed to any one of the three concrete mixers by

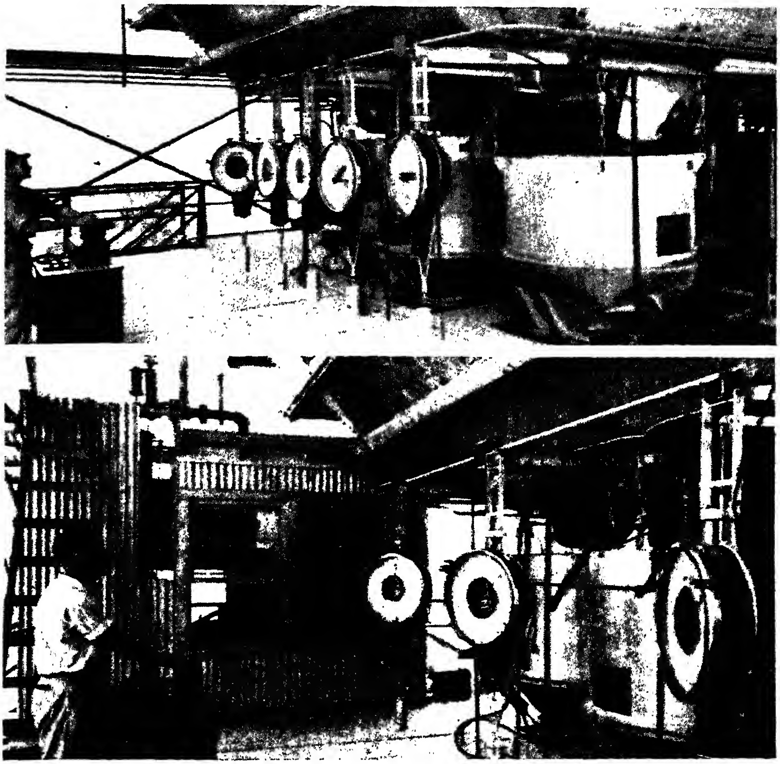


FIGURE 125.—*Batcher stands.*

means of a two-horsepower, motor-driven turnhead. An auxiliary turnhead attached to the main turnhead handled mixing water.

The three mixers were powered independently by 440-volt, three-phase, 1,160 revolutions per minute, 40-horsepower motors. Each mixer was mounted on a separate frame and was connected to its motor by means of a tex-rope drive. Each mixer was tilted independently by an air cylinder controlled by an air control valve. The mixers were, in general, of standard design with the exception that a coating of hard-surfacing metal was applied to the interior to resist abrasion. The coating was $\frac{5}{16}$-inch thick where the wear was the

heaviest and tapered down to 1/8-inch where the least wear occurred. The coating was kept built up rather than waiting until large areas were worn off. The coating was put on with special welding rods of nickel and cast iron composition and was applied by means of reverse polarity electric welding. The only points of wear were the extreme upper edge of the blades and the area where the concrete struck the drum shell after falling over the blade. To prevent the blade edges from becoming irregular in shape after wearing, a 2-inch plate was welded to the back of the blade at the upper edge. The coating was then added to the edge of the blade to bring it to the top level of the added strip.

FIGURE 126.—*Concrete mixers.*

The mixer center lines were set radially so as to discharge through a common hopper into cars on a track at the level of the base of the mixer building.

The three mixers were controlled from a one-man C. S. Johnson & Co. operating stand placed on the mixer floor so that the operator could see each mixer and the product that it discharged. Koehring mechanical timers indicated the minimum mixing time of the material in each mixer. The mixers were designed for three minutes' mixing time. By test, however, it was found that 2½ minutes was a desired mixing time. This mixing time gave a mixing cycle of about 3 minutes and 15 seconds.

Raw water pumped to a storage tank on a hill above the mixing plant was used for mixing water, sand plant, and washing and curing of concrete in place. Two auxiliary tanks only slightly higher than the mixing plant water batcher were installed to reduce water pressure on the batcher valves. Mixing water was heated in cold weather by injecting steam into one of these auxiliary tanks. A 150-horse-

power, 125-pound, Pennsylvania portable locomotive-type boiler supplied steam for heating mixing water and for thawing out sand plant stock piles, conveyors, and other equipment. The boiler was erected on the slope between the cement silo and the mixing plant. Mixing plant water was heated only when concrete placing temperature was below 50° F.

Compressed air for operating batcher equipment, mixer dump mechanism, and other equipment was supplied by the job compressor plant aided by a compressor located near the mixing plant. Low air pressure caused operating difficulty at times.

Operation.

Operation instructions are given in full in figure 124. When aggregates were needed in the storage bins, the upper turnhead operator signalled the reclaiming tunnel operator (and his helper) by horn and signal light, the latter indicating the size aggregate needed. Only one size of aggregate could be run at a time, but sizes could be run in any sequence. The tunnel operator opened the gate in the roof of the tunnel, and the material flowed on to the tunnel conveyor belt, which in turn deposited it on the elevating belt. From this belt it passed through a turnhead into the respective bins.

The first of the two batcher operators weighed cobbles, coarse rock, medium rock, and cement, and then signalled with a bell when his part of the batch was weighed. The second operator weighed fine rock, coarse sand, and fine sand, and dumped all batchers when the batching was completed. He then closed all batcher dump gates after he had received a signal from the first operator that his batchers were empty. The material was dumped only when lights indicated that the turnhead was directed toward an empty mixer. Lights on the water batcher showed when all inflow and outflow had ceased. A glass gage was also provided as a visible check against the lights. Water was automatically batched to the amount set by the dispatcher and was dumped by the second batcher operator.

The dumping lights mentioned above consisted of an automatic light which indicated the direction of the turnhead discharge, and a manually-operated pair—one for the mixer operator and one for the batcher dumping operator. This latter light was controlled by two-way switches and was turned out by the batcher dumping operator after the batches were emptied. It was lighted by the mixer operator when the mixer was ready to be charged. At the completion of a charge, the turnhead was moved to the next mixer by the mixer operator. The moving of the turnhead automatically started a mechanical timer which prohibited dumping of a mixer before a minimum predetermined mixing time had elapsed. The mixer operator emptied the mixer through a hopper into one of two dump bodies on the haulage car. Train operators were signalled by horn to move up into a position to load, and to proceed to the cableway loading trestle.

Two clean-up men, who acted as relief operators, were assigned to the plant. An oiler kept the moving parts of the mixing plant oiled and greased, did the greasing on the locomotives, and assisted in cleaning the mixers. A mechanic made minor repairs, contacted

the mechanical department for major ones, and acted as operating foreman of the plant. Electrical maintenance to the mixing plant and to the cableway head towers was cared for by an electrician, who spent most of his time at the mixing plant.

The complete crew per shift of the mixing plant was:

Number	Classification	Duty	Rate of pay
			Per hour
	Machinist foreman	Foreman	$1.25
	Electrician		1.00
	Operators	Mixer and two batchers	.75
	Operator	Upper turnhead	.75
	Helpers	Relief operators and plant clean-up	.60
	Oiler	Mixing plant and locomotives	.60
	Operator	Reclaiming tunnel	.75
	Helper	do	.60
	Fireman	Water heating plant—cold weather only	.75
			Per annum
	Concrete dispatcher	Operation and inspection	$2,000.00
	Mixing plant inspector	Inspection	1,620.00

Dispatching was handled from a small control room on the batcher floor. This room was equipped with a control board, water batcher control, signal switches, telephones, apparatus for determining moisture content of aggregates, and other equipment. By means of the control board, five different concrete mixes could be produced without resetting the weigh-batcher scale indicators. Five sets of pointer-lights on weigh-batcher scale dials operated from the control board made rapid mix selections possible. Batch counters for each of the five mixes furnished record data. Signal lights controlled by the dispatcher routed transfer trains to the proper cableway. After development of operating technique, it was found possible to deliver several kinds of concrete to two or even three locations in the dam without confusion or delays. Success of such complicated operation was due almost entirely to the dispatchers who actually controlled the entire mixing plant and transportation operations.

Responsibility was placed upon the dispatcher for taking orders from the inspectors in the forms for concrete and controlling the plant operation to deliver the correct mixes as desired. Orders for concrete were received by telephone. By operation of mix selector switches on the panel the pointer-lights for the desired mix became illuminated, thus indicating the amount of each material to be batched. The dispatcher used light signals to designate to which cableway the train was to carry concrete. A shift report was prepared by the dispatcher showing the amount of concrete poured, where placed, the amounts of different materials used, and delays. Copies were made for the job, main office, and warehouse.

The mixing plant inspector acted as a relief dispatcher, helped in readjusting the pointer-lights for mix revisions, and checked the amount of moisture in the aggregate. He also computed the amount of water needed for each mix. He set markers on the water batcher for convenience of the dispatcher in setting the batcher overflow, and was responsible for checking the mechanical timers.

Production.

Performance of the mixing plant as a whole was satisfactory. Mixing of concrete was begun on July 17, 1934, and completed in June 1936. Periods of maximum production on a batch count basis were:

	Cubic yards	*Cubic yards per hour*
Month, April 1935	92,780	157.3
Day, January 7, 1935	4,090	170.4
Shift, July 10, 1935	1,446	180.7

The unit mixing cost per cubic yard of concrete was $0.282. An approximate breakdown of the unit mixing cost is as follows:

	Cost per cubic yard
Heating plant	$0.013
Aggregate reclaiming plant	.079
Mixing plant	.190
Total mixing cost	.282

Another breakdown of the unit mixing cost is given in figure 127. For the maximum month, April 1935, this unit cost was reduced to $0.231 per cubic yard.

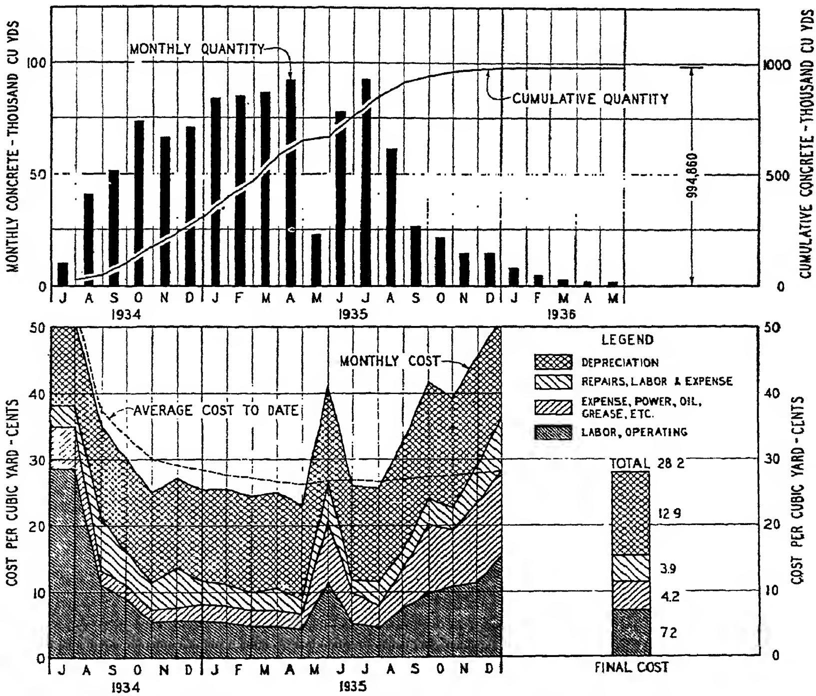


Figure 127.—*Concrete mixer plant—Quantities and costs.*

The initial cost of the concrete mixer plant erected, including aggregate reclaiming tunnel and reclaiming conveyor and structure, is given as follows:

Aggregate reclaiming plant	\$52, 689. 67
Mixing plant	83, 719. 15
Water heating plant	4, 797. 72
Total concrete mixer plant	141, 206. 54

Changes in installation and repairs.

Two of the mixer drums were returned to the manufacturer and replaced because the tracks around the drum wore unevenly. One mixer was returned after having mixed about 66,000 cubic yards of concrete and the other after having mixed approximately 186,000 cubic yards. Gear castings of two of the mixers were replaced after having broken and in each case were replaced by the manufacturer.

A recording wattmeter for each mixer was installed on the control panel for use as a consistency meter. They were not quite suited for this use but were extremely useful in furnishing information to the dispatcher, such as approximate mixing time, records of batches mixed, unusual variations in consistency, and in showing which mixers were charged.

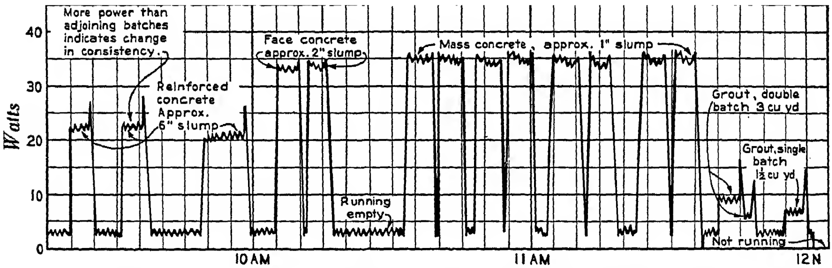


FIGURE 128.—*Part of recording wattmeter chart (consistency meter).*

In the original installation water was put in at the rear of the mixer and was directed toward the dumping end. This arrangement caused poor mixing and varied consistencies in the mix. It was changed so the water, upon entering, was directed to the right and hit the blades as the mixer turned toward it.

Two four-vane rotary feeders, one between each cement bin and batcher, were installed, but these were changed to eight-vane feeders before satisfactory operation was obtained.

The sand batcher did not dump cleanly when the sand was wet. Since this was the usual condition of the sand a small vibrator was installed. This did not prove entirely satisfactory and probably a larger vibrator would have been more satisfactory.

Soon after installation, the valves of the water batcher began to leak, due largely to the high pressure on the line. When a low pressure water supply was provided, leaking continued but in smaller quantities. As originally installed, the slope of the discharge pipe

from the water batcher was too flat and the pipe from the turnhead too small. The slope of the discharge pipe was made greater, and pipes from the turnhead were increased to 12 inches for part of their length and then reduced to the original 5-inch diameter. Hand setting of the water batcher was provided with the original installation, but this method was too slow and a motor drive was installed. The motor had a flexible shaft connecting it with the batcher overflow adjustment which was changed to a telescoping type because of excessive breaking.

Automobile-type bulbs used for the scale pointer-lights were continually breaking. A screw-type bulb would be better. Also the pointers were so wide that it was impossible to set two of them on weights very nearly the same. A narrower pointer would remedy this situation.

Heat should be provided around all bin doors and possibly in the sand bins to avoid freezing up in winter. In similar climate the batching room should be enclosed and heated.

Interlocks should be provided so the operator could not dump water until the required amount was in the tank and all inflow and outflow had ceased.

TRANSFER TRAIN SYSTEM

Concrete was transferred from the mixing plant to cableways in transfer cars especially designed for the purpose. One gasoline-electric locomotive handled each transfer car. Three transfer cars and four locomotives—one spare—were provided. All three 1-car trains were operated during periods of high concrete production.

The four locomotives were purchased from the Plymouth Locomotive Works. These locomotives weighed 9 tons each and had a pulling capacity of 30 tons. They were powered by a Buda 92-horsepower gasoline engine through a Westinghouse 50-kilowatt, direct-current generator and a 52-horsepower motor. Their safe maximum running speed was 23 miles per hour. Specifications for performance called for a starting tractive effort of 5,000 pounds with 25 percent adhesion and a continuous tractive effort of 1,650 pounds.

The concrete transfer cars were built to the Authority's specifications by the Insley Manufacturing Co. Mounted on each car there were two dump bodies which faced each other and discharged into a central hopper which, in turn, discharged into a chute. Each dump body was operated by a 15-horsepower motor which received its power from the generator on the locomotive. Control equipment for the transfer cars was installed on the locomotives. Each dump body had a 3-cubic-yard capacity, making the total capacity 6 cubic yards.

Plant lay-out and operation.

The mixers, at about elevation 1,070, discharged the transfer cars which ran the full traverse of the cableways on a standard gage 60-pound rail track. A loading platform ran practically the entire length of the track and was used as a landing for the concrete buckets while they were receiving concrete from the transfer cars. This platform was constructed below the level of the tracks to permit gravity transfer of concrete from the car to the bucket. A double

track led from the mixer plant along the loading platform, and three crossovers were provided to facilitate passage. The maximum one-way distance of travel for any load was approximately 750 feet.

The mixers, operating on 3-minute cycles, all emptied through a central discharge chute in consecutive order so that each train required about 2 minutes to receive its load.

Concrete trains approached the mixer plant from one direction only and stopped with the forward dump body under the discharge chute. When this dump body was filled, a signal from a horn controlled by the mixer operator notified the train operator to bring the rear dump body into place. A second signal was a notice to move

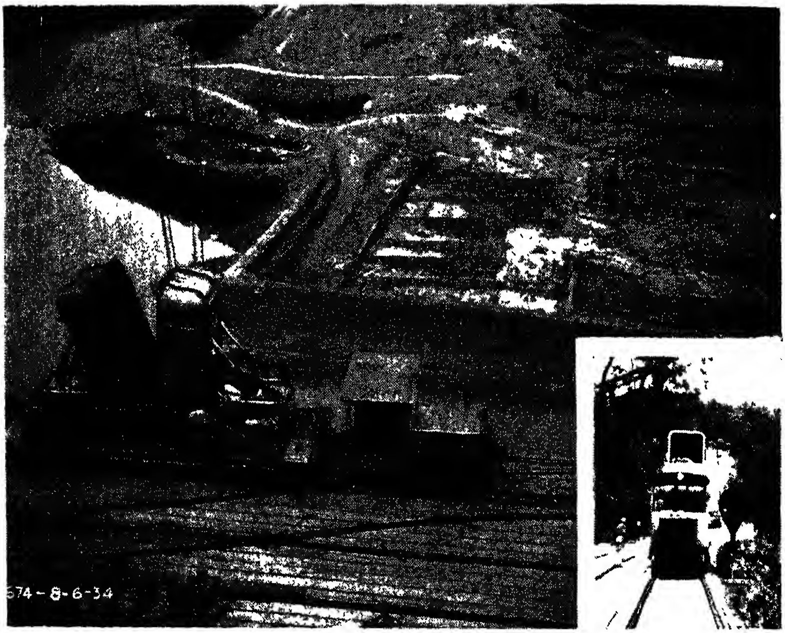


FIGURE 129.—*Transfer cars.*

out with the loaded car. As the train left the mixer plant, the third mixer was ready to discharge into the next train as soon as it was spotted under the chute.

washed out and oiled the dump bodies at proper intervals to prevent concrete from hardening on the surfaces. Each of the three transfer trains was operated by one man at 75 cents per hour. Direct peak operations gas consumption averaged 30 gallons per locomotive per operating day.

Job experience showed that the rate of placing concrete was limited by the capacity of the concrete plant rather than by the speed of the transfer equipment or the cableways. The figures of maximum production given in the discussion of the mixing plant were the maximum rates handled by the transfer cars.

Maintenance, repairs, and additions.

After some use of this equipment, it was discovered that the mechanical braking system was insufficient to stop the train properly when fully loaded. An air braking system was designed and installed on the job to remedy this trouble. It consisted of standard Westinghouse railroad air brakes on the front trucks of the transfer cars and standard Bendix-Westinghouse, diaphragm-type, automotive brake chambers to actuate the original mechanical brake arrangement on the locomotives.

The abrasive action of the concrete wore the surface of the dump bodies, hoppers, and chutes rather rapidly. The greatest wear in the dump bodies occurred in the impact areas where the concrete was received and the lips over which it passed in dumping. The greatest wear on the hoppers was about one-third of the way from the top where the mass of the concrete met the walls. Chutes wore at the point where the bulk of the concrete met the trough and at the point of departure. All worn surfaces were reinforced with ⅜-inch steel plate welded in place.

Originally a rope hoist which lowered the discharge chute was manually operated from the locomotive cab, but this was later modified by attaching the rope to the forward dump body in such a manner that its dumping automatically lowered the chute. When the first dump body had discharged its load, it was returned only partially, [illegible]ermitting the chute to remain down, while the second dump body [illegible]as discharged.

A flat-braided cable, located in a channel bent to form a circular [illegible]rc and connected to the dump body hoisting motor drive had a ten[illegible]ency to break at the point of connection to the dump body. This was [illegible]tially remedied by placing a metal strap across it at that point.

[illegible]ertain weaknesses in the lay-out of the concrete hauling system [illegible]h became evident once operations were under way, but owing to the [illegible]ography of the site not all could have been eliminated. A track [illegible]out curves would be desirable to permit clear vision for the op[illegible]or when backing. A few minor collisions resulted from the oper[illegible]'s inability to see well under such conditions. To minimize delays [illegible] misunderstandings, all trains should be routed through the mixer [illegible]nt in the same direction, using a track around the mixer.

[illegible]n the whole, the concrete haulage system worked well, the con[illegible]uction forces being generally satisfied with its performance. The [illegible] mechanical troubles and modifications have been explained. [illegible] these weaknesses were remedied no further difficulty arose, and [illegible] performed reliably and efficiently.

Cost.

The cost per cubic yard of concrete, based on 989,299 cubic yards handled by the transfer cars and cableway buckets, is shown in figure 130. The cost per cubic yard of concrete handled by transfer cars is made up briefly as follows:

	Total cost	Cost per cubic yard
Labor operating	$33, 745. 97	$0. 034
Miscellaneous expense	20, 718. 17	. 021
Repairs	42, 956. 18	. 043
Depreciation	55, 508. 57	. 056
Total	152, 928. 89	. 154

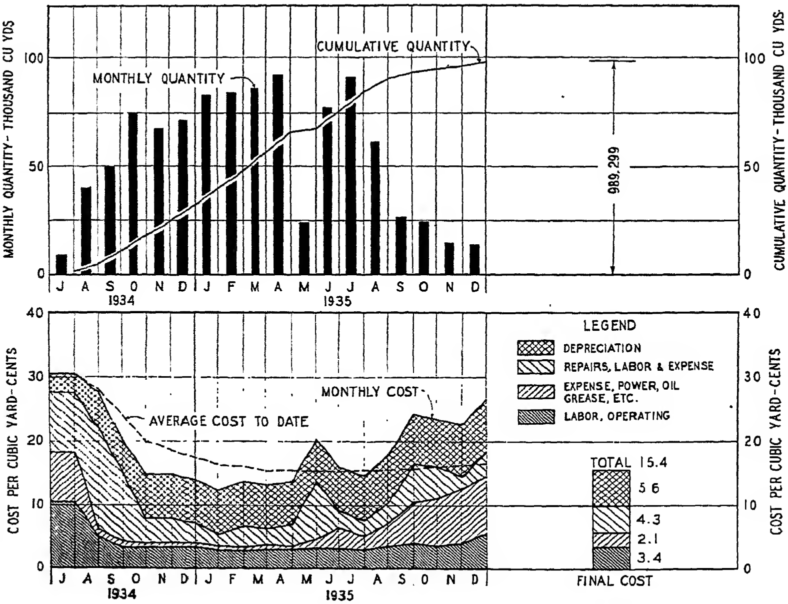


FIGURE 130.—*Concrete haulage—Quantities and costs.*

CABLEWAYS

Concrete was carried in buckets from the transfer track loading platform to any desired location at the dam or powerhouse by two traveling, thrust-wheel type cableways. Flexibility of operation was such that after operating technique had been developed, it was possible to place concrete in one location with both cableways simultaneously without delay by synchronizing traversing of the cableway towers. Originally expected to handle 3,000 cubic yards of concrete per day, the two cableways handled 4,000 cubic yards with ease even over long hauls. With short hauls, one cableway placed

concrete at a rate of 140 cubic yards per hour for extended periods.

The cableway method of concrete placing was well adapted to the conditions at Norris, and the design and operation of the equipment installed was remarkably satisfactory from the standpoint of both concrete quality and cost. Unfortunately, combinations of circumstances did not permit operation of both cableways at full capacity; and their full value can be estimated only from performance under the conditions imposed. In comparison with other more complicated systems, the results were most gratifying. In combination with the transfer system employed, the cableways permitted delivery of concrete to its final position in the form with a minimum of segregation. In addition to the main purpose of placing concrete, the cableways handled a large portion of the heavy permanent equipment in the dam, closure gates, forms, pipes, and other items, to locations difficult of access by other means.

Previous cableway experience.

Norris Dam was the third construction project to use heavy-duty cableways supported on traveling towers especially constructed to take the horizontal thrust entirely through a set of horizontal thrust

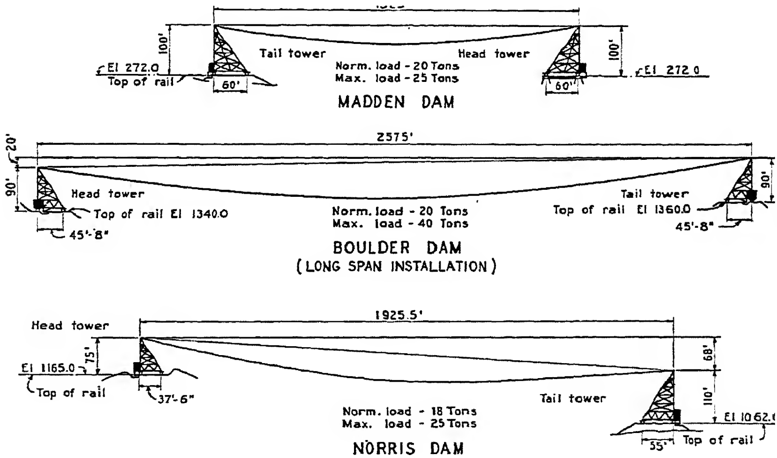


Figure 131.—*Comparison of recent cableway installations.*

wheels. The first project to employ this type of heavy-duty cableway was Madden Dam in Panama. Soon after the successful use of the Madden Dam cableway, three similar installations with spans ranging from 1,365 feet to 2,575 feet were made at Boulder Dam.

Previous to these three installations on projects where traveling-type cableway towers were used, towers were usually mounted on the heaviest type of railroad track, and the entire horizontal thrust was carried by the wheel flanges. As the projects grew in size, the loads were increased and the number of vertical wheels necessary to support the towers and at the same time carry the horizontal thrust

through the wheel flanges was greatly increased. This excessive flange load caused a great many broken axles and, with the resulting shutdown in work while repairs were being made, caused additional expense.

The thrust-wheel type cableway permits the entire horizontal thrust to be placed on the back side of the runway; whereas, on the vertical flanged-wheel type cableway as high as 80 percent of the thrust is taken by the front edge of the runway under full load. The use of thrust wheels at Norris permitted the earth-fill runway at the tail tower to be built 10 feet narrower than the width which the conventional type would have required. In the case of the head tower runway, it would have been dangerous to permit thrust on the front of the runway because this was directly on the edge of the bluff. This was especially true where trestles were necessary at the ends of the head-tower runway. For these reasons towers with horizontal thrust wheels at the back of the runways were chosen. This type of design later proved its effectiveness when a "cave in" occurred under the head-tower runway. It was conceivable that one of the cableways might have been badly damaged had not the horizontal thrust been transmitted to the back of the runway where it was solidly anchored into the hill.

The 6-cubic-yard buckets used with a cableway span of 1,925.5 feet would cause a horizontal thrust on the towers of over 200 tons, obviously too great to carry on wheel flanges. A thrust wheel type tower was clearly necessary under these conditions, as the use of smaller buckets would require a longer time to build the dam, and the cost of placing concrete would be correspondingly higher.

Preliminary investigations.

In preliminary investigations several methods of placing concrete were considered. Both the use of a high trestle with whirley cranes for placing concrete and the use of a derrick installation were studied. Although the initial cost of these alternate schemes was considerably cheaper than the cableway scheme, the large quantity of concrete to be placed justified a large capital outlay, provided sufficient practical advantages for the cableway made it desirable. The following advantages were considered sufficient to justify the increased cost of the cableway installation:

1. Clear access was provided to all parts of the work at all times. Trestles and derricks are often in the way.
2. A hoisting capacity of 18 tons was provided at all parts of the job at any time, and a load could be transferred easily from any part of the job to any other. Loads up to 50 tons could be handled by using two cableways, and making special provision for careful handling to avoid impact.
3. Penstock sections, powerhouse steel, drum gates, roadway bridge, form panel assemblies, and other miscellaneous equipment could be transferred and set in place without making special derrick set-up or otherwise making special provision.
4. A short span and favorable runway topography brought the cableway cost to a minimum for such installation.
5. The necessity for continually shifting derricks, changing guy wires, and altering and removing trestles was eliminated.

Equipment.

The cableways were purchased in January 1934. Several manufacturers cooperated in furnishing the complete equipment. Lidgerwood Manufacturing Co., Elizabeth, N. J., furnished the cables, carriages, hoists, motors, and control equipment; while the towers were contracted by the Virginia Bridge & Iron Co., Roanoke, Va. The original installation of operating cables and track cables was manufactured by the American Steel & Wire Co. Electrical equipment was manufactured by the General Electric Co. Trucks for the towers were manufactured by the Whiting Corporation.

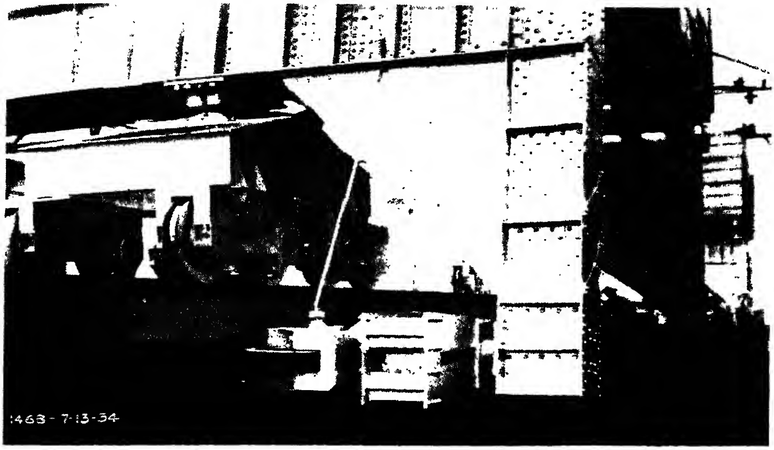


FIGURE 132.—*Thrust rail and ball and socket truck connection.*

Bids were opened on December 6, 1933, and an exhaustive and detailed analysis was made. The bidders' designs of the steel towers and loads and stresses were carefully checked. A detailed comparison was made of the hoists, motors, and all mechanical and electrical equipment that were offered, and all details were carefully compared. In analyzing the bids, a comparison of the cost of runways was made with the various types of towers submitted. In some cases the design of the towers themselves necessitated extra runway cost to provide for the horizontal thrust. Final analysis was made on the basis of the total cost of the entire installation including runways.

Design data.—Among the noteworthy features incorporated in the design of these cableways were thrust wheels on each tower to take up the horizontal thrust, a ball and socket connection between the trucks and the tower frame, and trucks designed to equalize and distribute the load evenly on all wheels. This latter feature provided for the even distribution of weight on the truck wheels at all times even though the runways might settle and get out of line. The value was clearly demonstrated when the green fill of the tail tower runway settled as much as 5 inches without causing damage to the structure or truck axles.

Each cableway was designed to operate as follows:

Full load	tons (plus 25 percent for impact)	18
Load at continuous operation	tons	16
Lowering speed at full load	feet per minute	400
Hoisting speed at full load	do	300
Carriage travel at full load	do	1,200
Traversing speed of towers	do	50

Regeneration took place when lowering the load, and when running down slope with the carriage. It was not possible to raise or lower the load with the carriage in motion. However, normal operations could continue while the towers were traversing.

Each cableway consisted of a head tower located on the west bank of the river and a tail tower on the east bank, with a horizontal span of 1,925.5 feet. The towers were connected by a track cable upon which ran a carriage, and suspended from the carriage was the main fall block or hook. The suspension point of the tail tower was 68 feet lower than a similar point at the head tower. The towers of each unit ran on parallel tracks perpendicular to the dam axis, and operated simultaneously at the same speed. The lay-out of the job was such that from the point at which the cableways received the concrete, haulage was generally downward until it was placed in the forms.

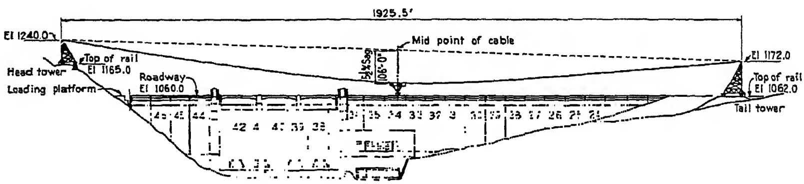


FIGURE 133.—*Cableway lay-out.*

Towers and runways.—The head towers were located on the west side of the river. The vertical section of the tower was a right angle with the base level, and the back leg vertical. This vertical side is 75 feet from the rail to the suspension point of the track cable. Each tower was designed to withstand the horizontal pull of the track cable, operating cables, and light and conductor cables. These horizontal pulls were calculated to be:

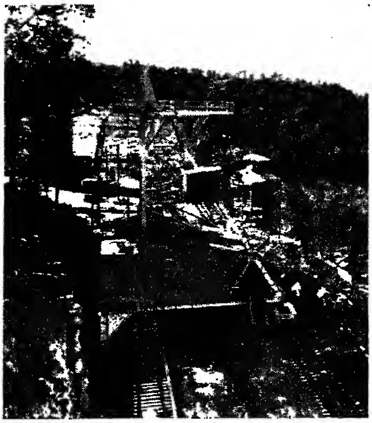
Track cable pull, 18-ton load midspan	362,000 pounds
Operating cable pulls	31,000 pounds
Light and conductor cable pulls	60,000 pounds
Total without impact	453,000 pounds

In addition, impact equal to the stress added by increasing the load on the track cable 25 percent was provided for. The head tower carried 390 tons of counterweight as well as the operating cab, main hoist, traversing equipment, and electrical and operating equipment. The counterweight was designed equal to 25 percent more than the weight required to balance a 25-ton load at midspan. The tail towers were similar to the head towers in the vertical section, having a height of 110 feet from the runway rail to the suspension point of the track cable. They were designed for the same horizontal pull

as the head tower and carried 445 tons of counterweight, as well as the traversing motor. Two lines of floodlights were suspended between the head and tail towers of each unit. This floodlight system consisted of 34 1,000-watt lights, 17 on each cableway.

Power was supplied at 2,200 volts to a 3-wire trolley in the rear of the towers. General views of both head and tail towers are shown in figure 134.

Cables and carriage.—A 3-inch American Steel & Wire Co. locked coil type track cable, 1,900 feet long between sockets, was stretched with a 5½ percent sag between the suspension point of the head and tail tower of each unit. Under a full load of 18 tons, a tension of 362,000 pounds excluding impact was developed in this cable. The



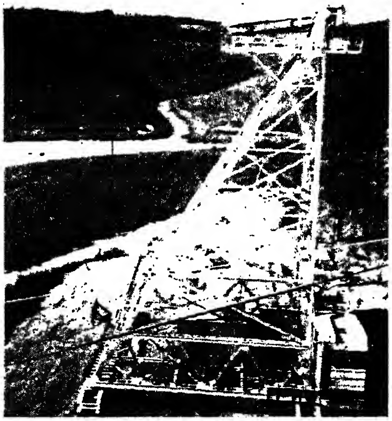


FIGURE 134.—*Head and tail towers.*

sag of the track cable provided 40 feet clearance above the top of the dam when under full load. At the tail tower a 1⅜-inch take-up cable reeved as a 10-part line connected the track cable to the tower and permitted the sag in the main cable to be adjusted when necessary.

A carriage mounted on twelve 24-inch roller sheaves ran on the track cable and carried the main fall block and hook. It was made up of two sections, the main carriage and the auxiliary carriage. Each section was supported on six 24-inch diameter roller-bearing track wheels. These wheels were equalized to provide as nearly as possible equal loads on the track cable. The carriage supported a 25-ton hook mounted on a swivel bearing on the fall block. This fall block contained two 44-inch diameter sheaves mounted on roller bearings. Each section of the carriage supported a sheave to carry the hoist line. The reason that these sheaves were placed separately in each carriage, instead of side by side in one main carriage, was twofold: (1) This arrangement spread out the load in such a way that more track wheels could be used, thus reducing the maximum wheel load on the track cable; and (2) it provided a means of pre-

venting the hook and fall block from spinning. With this arrangement the bucket always faced the same way and did not spin in the air.

For moving the carriage back and forth along the track cable, a 1-inch endless line 4,110 feet long was used. The hoist line was a

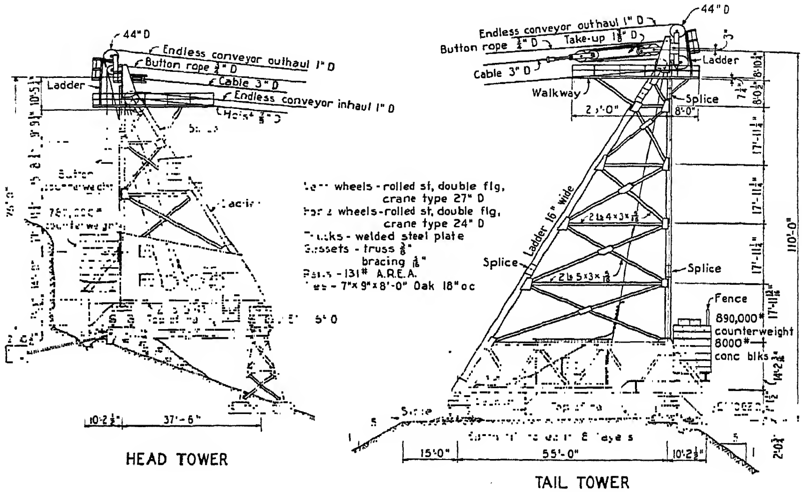


FIGURE 135.—*General arrangement of the towers.*

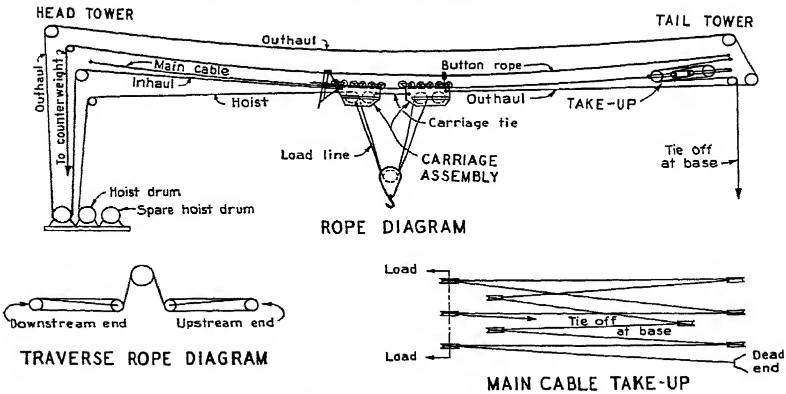


FIGURE 136.—*General arrangement of cables and carriage.*

⅞-inch cable 2,500 feet long. It ran from the hoist drum at the head tower to the carriage, where it raised or lowered the load by a four-part line on the fall block and hook. The button line was a ¾-inch cable, 2,025 feet long. Twelve buttons were located on this line. They varied from a diameter of 1⅝ inches near the head

tower to $3\frac{9}{16}$ inches near the tail tower at intervals of 170 feet across the span. The purpose of this cable was to distribute the slack cable carriers at even intervals across the cableway span.

To move the towers perpendicular to the dam axis, each tower had 2,100 feet of $\frac{7}{8}$-inch traversing cable forming a four-part line which passed through blocks at each end of the tower runway. This cable made four turns on the endless drum of the traversing winch and dead-ended at the tower.

Operating equipment.—The main hoist consisted of three drums mounted in tandem and connected by separate clutches to the gear train which was in constant mesh with the motor spur gear. One drum was used for the endless line, the second for the hoist line, and the third, which on this cableway was not utilized, could be used for a spare. The endless line made four and one-half turns on the movable wearing plates of the endless drum. As originally equipped, the three main clutches were furnished to operate through a magnetically controlled air valve. This arrangement was later changed to a direct air system. Each drum was also equipped with a V-brake, weight set, and released by a magnetically controlled air valve. Air was supplied by two 50 cubic feet per minute compressors for each tower. A 400-horsepower variable speed induction motor was used to drive the main hoist. It operated at six speeds in either direction and was equipped for regenerative breaking and full stop under load. For traversing the towers, a 75-horsepower variable speed induction motor was used.

Safety features.—The following safety features were incorporated in these cableways:

1. An overspeed tachometer from each drum automatically set the brakes in case of a runaway.
2. No-voltage relays set the brakes in case of a power failure.
3. The third drum of the main hoist, not used at Norris, was permanently connected to the hoist drum to allow the use of both brakes and frictions in the hoisting operation.
4. Limit switches on the towers made it impossible for the towers to collide or run off either end of the track.
5. Skew limits prevented the head tower or tail tower of one cableway from getting more than 40 feet out of line.
6. The original design of the cableway provided for dumping the buckets from the cableway tower. This idea was abandoned, however, because of the possibility of the danger of accidentally dumping the bucket in midair.

Tests.—In order to confirm design assumptions, several tests were made and considerable data secured which in general check the results given by the design formulas, and in particular point the way to certain precautions to be used in the application of these formulas. Tests with a Martin-Decker heavy-duty tension indicator were made to determine actual stresses in the button cable, the outhaul cable, the carriage tie, the hoisting cable, the tower traversing cables, and the 10-part take-up cable, the latter being translated into actual stresses in the main track cable.

Before the cableways were placed in operation a series of load tests was made. The test load was conveyed, lowered, and raised, and the towers were traversed. The brakes were severely tested by

allowing the load to run away and be caught by the brakes. Safety devices were checked, and the entire installation carefully inspected.

Operation.

The system of operation of these cableways provided for the location of the operator in a control cab where he could see the work and observe the performance of the cableway in handling its load. He also had the further advantage in this case of seeing the tail tower. All operations were by remote control.

The operator acted on instructions received from one of two signalmen by telephone through a loudspeaker in the cab. One signalman was located at the loading point to control the loading and dispatching operations, while the second signalman was at the discharge point and controlled operations there. All three men were in two-way communication at all times. Every operation was ordered by a signalman, and the operator took matters in hand only in emergencies. The value of the visual control system in preventing serious accidents was demonstrated several times on this job.

The concrete bucket was of the bottom-dump type with a water level capacity of 7½ cubic yards. Normally, however, only 6 cubic yards of concrete were carried to keep within the capacity of the cableway. The bucket was especially designed to reduce the "bounce" when it was dumped and in turn lessen the vertical oscillation and impact on the track cable.

To cause the filled bucket to swing clear of the transfer tracks, trucks, and trestle, the carriage was run a few feet outward just prior to hoisting. The bucket was then hoisted high enough to clear all obstructions to the point of dumping and the carriage run out. The carriage was brought to a stop in such a way as to eliminate swing of the bucket. After lowering and dumping, the bucket was again raised to clear obstructions, and the carriage was returned to the loading point. The bucket was lowered to the loading platform as it swung slightly, to permit the best landing.

This cycle could be completed to any spot within range of the cableway within 4 minutes. At times at close range it was made in 2 minutes. Efficient cableway operation depended not only on the cableway operator, but also on the locomotive operator who spotted and dumped the transfer cars and on the dump man in the forms.

During peak production, four crews each worked 6 hours. The cableway operated 23 hours per day, 1 hour being taken for inspection and greasing. In addition to the general cableway foreman, the normal operating crew necessary for two cableways was:

Number	Classification	Rate of pay per hour	Total cost per hour
2	Operators	$1.50	$3.00
1	Head tower oiler	1.00	1.00
1	Tail tower oiler	.60	.60
4	Signalmen	1.00	4.00
1	Foreman	[1] 1.00	1.00
1	do	[1] 1.50	1.50
Total			11.10

[1] Acted as spare operators when necessary.

Most of the operators had had previous cableway experience before their employment at Norris. The signalmen and the oiler in the head tower were riggers who showed special ability in their respective jobs, while the tail tower oiler was a semiskilled laborer. Each of the foremen was a former operator.

Performance.

Concrete constituted about 97 percent of the tonnage carried by the cableways. In addition, they were used to handle the following equipment and materials to April 1, 1936:

Item:	*Weight in tons*
Construction machinery	[1] 210
Dam closure operation	223
Dismantling concrete, cofferdam No. 3	3, 850
Drum gate steel and machinery	680
Excavation	[1] 22, 500
Form hoists	[1] 11, 250
Form lumber	[2] 3, 400
Grout cement	[1] 4, 840
Penstock steel	586
Pipe and conduit	[1] 500
Powerhouse permanent equipment	1, 760
Powerhouse structural steel	810
Reinforcing steel	[1] 2, 800
Riprap, east embankment	8, 190
Road material and maintenance	[1] 875
Spillway bridge steel	375
Outlet conduit equipment	1, 087
Tile and rock for drains	900
Timber cribs for cofferdam	[1] 4, 375
Tractor gate and operating machinery	385
Trashrack steel	575

[1] Estimated.
[2] Based on assumption that cableway handled forms twice; once when they were moved to the block, and once when they were no longer needed.

Material that came in by truck was taken from a road that extended to the foot of the dam next to the powerhouse on the east side of the river. For such unloading, the cableways traveled to a point downstream from the dam. Since the carpenter shop was located within the range of the cableways, forms built there could be moved directly to the point at which they were to be used.

The maximum placement of concrete in a single month for both cableways was in April 1935, with 92,780 cubic yards. In this period the average hourly placement was 157.3 cubic yards for both cableways. The maximum 24-hour placement was made January 7, 1935, when 4,090 cubic yards were placed for an average of 170.4 cubic yards per hour. The maximum rate in a single shift was made July 10, 1935, when 1,445.8 cubic yards were poured between 4 and 12 p. m. for an average of 180.7 cubic yards per hour. Under perfect conditions one cableway placed 162 cubic yards per hour, but normally 120 cubic yards per hour could be placed at continuous operation.

The average concrete-placing cycle remained about constant for both high and low placement operations. This was due to the fact that when placing in higher blocks the bucket had to be raised high enough to clear all obstructions, while in the early part of the job the bucket could be operated at the level of the loading platform in traveling to and from the forms.

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The cableways were limited to 180 cubic yards per hour, the capacity of the mixing plant operating on a 3-minute mixing cycle, which was more or less the limiting condition of the entire plant. Minor delays to concrete placement were caused by one cableway making several lifts, such as lumber or form hoists. It was a practice to stagger the time of these lifts through a shift, as one cableway could be off 10 minutes and come back on to clear the accumulated concrete from the loading track without delaying the mixing plant. The same condition was encountered in changing from one form to another. Here again one cableway carried more than its share of the concrete until the crew of the other had moved to another form and was ready again to begin placing.

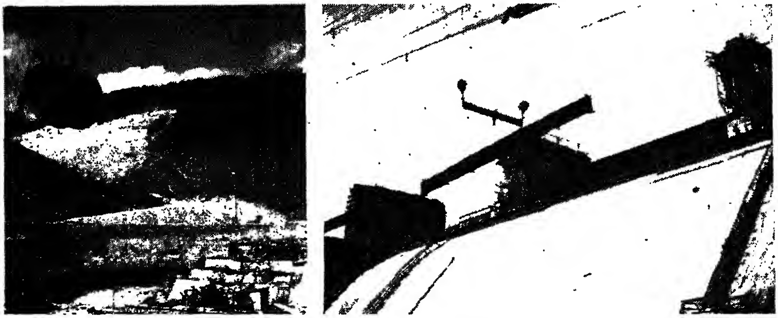

FIGURE 137.—*Both cableways handled heavy pieces of equipment.*

A summary of the performance of these cableways in the handling of concrete and other materials from July 17, 1934, to April 1, 1936, is shown in table 72.

TABLE 72.—*Cableway performance*

	Cableway No. 1	Cableway No. 2	Total
Elapsed hours	14, 496	14, 399	28, 895
Idle	3, 713	4, 243	7, 956
Gross operating hours	10, 783	10, 156	20, 939
Delays	998	1, 107	2, 105
Net operating hours	9, 785	9. 049	18, 834
Placing concrete	7, 295	6, 085	13, 380
Total other work	2, 490	2, 964	5, 454
Quantities handled:			
Concrete (cubic yards)	577, 840	427, 090	1, 004, 930
Other work (tons)	35, 010	33, 272	68, 282
Concrete (cubic yards per net hour)	79. 2	70. 2	149. 4
Other work (tons per net hour)	14. 1	11. 2	25. 3

Delays listed in the above table amounted to 9.2 percent and 10.09 percent of the gross operating time for cableways Nos. 1 and 2, respectively. Each cableway consumed, exclusive of lighting, an average of 66.2-kilowatt hours of electricity per gross operating hour.

Although each unit operated separately under ordinary conditions, both were used for placing loads in excess of the capacity of one

cableway, but within the combined capacity of both. This arrangement was used to place the following material:

	Weight of load (tons)	Amount placed
Penstock sections (20 feet diameter by 20 feet long)	36–40	586 tons.
Powerhouse crane girders (62 feet 3 inches long)	33	2 girders.
Generator rotor	35	1 unit.
Generator stator frame sections	30	8 sections.
Turbine water wheels	51	2 units.
Spillway bridge girders (107 feet long by 7 feet 1 inch deep)	40	6 girders.

The turbine water wheels constituted the heaviest load handled by the cableways. This load was lifted about 1 foot and carried about 18 feet, utilizing a lifting beam. The 48-ton wheel, together with the 6,800-pound lifting beam, made a total load of 51 tons or 25½ tons per cableway.

Maintenance and repairs.

The largest recurrent repair item consisted of operating cable replacement and maintenance. Whenever possible operating cable changes were made on Sundays, but in many cases changes had to be made during normal working time. At first these changes took considerable time, but later they could be made in an average time of 1 hour for either a button or load line, and 3 hours for an endless line. Cables were inspected and greased every day during peak operations, but later they were inspected every second day and greased once a week. While this maintenance and replacement could be made with as few as four riggers, more were often used to speed up the operation. These men formed a separate rigger crew and were used for all similar operations on the job.

The track cable was turned one-eighth of a revolution about its horizontal axis every 6 weeks to present a new rolling surface to the carriage wheel. At first the button lines failed at a point in front of the buttons. This seemed to be caused by the slack carrier rebounding after striking the button and hitting the cable, causing undue wear at that point. This was overcome to a considerable extent by slipping a pipe sleeve about 18 inches long over the cable in front of the button so that much of the blow from the carrier was taken on it rather than on the cable.

Cable performance during the first 16,947 hours of operation is shown in table 73.

The only repair of major proportion was caused by the failure of four of the six main hoist -brake flanges on the main drum. Two failures occurred during the period when the operators were being broken in. These two drums were returned to the factory for repair. The other two failures occurred when the unit had operated 2,221 and 8,523 hours, respectively, and were repaired on the job. This repair consisted of cutting off the broken flange, with a cutting tool mounted on the main hoist frame, while the drum was being turned with a small air hoist. A new cast-iron flange was then bolted to the drum to complete the repair.

After the cableways had completed their work, the two remaining original brake flanges were cut from the drum and replaced with new

flanges. This was done as a precaution against breakage that might occur when the cableways were transferred to another job or sold.

In May 1935 a portion of the head tower track caved in as a result of displacement of the natural earth base which had become saturated and moved into a subterranean cavern. Repair work required the excavation of a large quantity of clay and the construction of a reinforced concrete bridge across the cave-in to support the tracks.

TABLE 73.—*Cable performance during first 16,947 hours of operation*

Line	Size and type	Number replaced	Average operating hours	Material cost per operating hour
Button	¾-inch regular lay	10	534.3	$0.591
	¾-inch flat strand	5	1,586.8	.286
Endless	1-inch regular lay [1]	7	1,149.3	.795
	1-inch flat strand	4	1,467.6	.888
Load	⅞-inch regular lay	16	887.8	.519
	⅞-inch flat strand	1	806.5	.851
Traversing (new only)	⅞-inch 6 by 19 regular	6	1,936.0	.333
Take-up	1⅜-inch 6 by 19 regular	2	5,625.0	.053
Endless drum	With regular lay	6	567.6	.577
Wear plates (partial record)	With flat strand (sets)	2	2,478.6	.129

[1] Does not include one cable that broke after 86 hours' operation.

NOTE.—After cableways had been operated for a considerable period, used endless and hoist lines were used to replace worn traversing cables. These hours are less than the total operating time of the cableway because the actual life of cable could not be determined after 16,947 hours of operation.

Except for the change in method of operating the main clutches, as mentioned previously, the original design was altered but slightly. A small change was made on the main circuit breakers, as they had a tendency to arc to ground. The timing of the automatic relay for the motor control was also changed.

For electrical maintenance, an electrician spent part of his time at the head tower and was always at hand at the mixing plant for emergency work.

From the experience gained in the operation of the cableways, the following recommendations were made by the job forces as guides in the future use of the equipment:

1. Use of pipe sleeves on button line in front of each button.
2. Use of flat strand operating cables for button line and endless line use.
3. Reversing endless line end for end to distribute wear on this cable more evenly.
4. Using longer than necessary load lines so that short sections can be cut off the drum end to distribute wear more uniformly at the point where the load line makes the second lap around the load drum.

In addition to the above items, the following points are worthy of consideration in the construction of future cableways of this type:

1. Use of chain or gear drive instead of traversing cables for moving cableway towers.
2. Runways skewed with the axis of the dam, permitting both cableways to work on the axis of the dam at one time.

3. Placing the outhaul sheaves of the endless line off center from the main cable to prevent the endless line from striking the other cables.
4. Reverse position of head tower and tail tower which would provide a straight-line pull on outgoing endless lines.

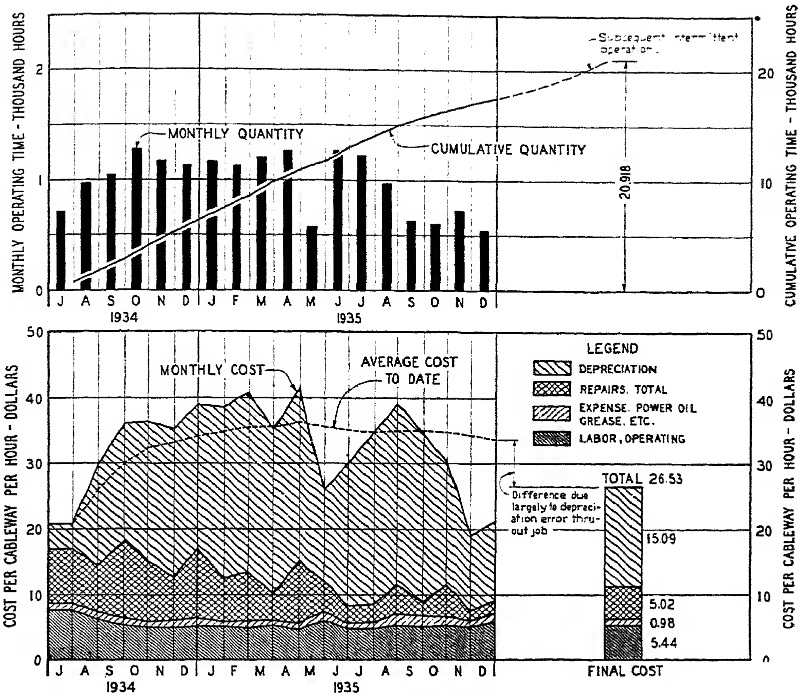

FIGURE 138.—*Cableway—Operating time and costs.*

Cost.

Operating cost for these cableways is shown in table 74.

TABLE 74.—*Analysis of cableway operating costs*

	Total	Cost per hour
Operation:		
Labor	$113,721.56	$5.437
Expense (power, air, lubricants, etc.)	20,431.10	.977
Total operation	134,152.66	6.414
Repairs and Maintenance	105,073.02	5.023
Total operating cost, less depreciation	239,225.68	11.437
Cost of reconstruction due to cave-in	63,723.25	3.046
Total per net hour (20,918 hours) operating distribution less depreciation	302,948.93	14.483

A summary of the cableway costs including depreciation, but without cave-in reconstruction costs, is shown in figure 138.

BUCKETS, FORMS, AND VIBRATORS

Buckets.

Three square 6-cubic-yard-capacity bottom-dump buckets, one 7-cubic-yard controllable bottom-dump bucket, and one 3-cubic-yard controllable side-dump bucket were used to transport the concrete from the transfer car loading platform to the forms. The bucket being used remained hooked to the cableway continuously during concrete operations.

The square 6-cubic-yard buckets were designed by the Authority and were made by the Insley Manufacturing Co. Before these buckets were constructed, a model bucket was made and tested in connection with the cableway design. Two buckets, whose inside dimensions were 5 by 5 by 7 feet, were delivered July 2, 1934, at a cost of $955 each. Each weighed approximately 6,900 pounds. As a result of job experience, several changes in design were made. A 12-inch lip was added and a pneumatic dumping arrangement replaced the manual arrangement of the first buckets. These changes were incorporated in the bucket delivered September 14, 1934, and increased its cost to about $1,090 and weight to 8,000 pounds.

The Blaw-Knox Co. designed and manufactured a controllable roller-gate, bottom-dump, 7-cubic-yard bucket. This bucket had an over-all height of 10 feet 3 inches, an outside diameter of 8 feet 6 inches, and weighed 6,250 pounds. The baffle and gate were set on an angle to insure vertical discharge. It was secured November 6, 1934, at a cost of $1,320.

A 3-cubic-yard side-dump bucket was made in the machine shop at the dam at a cost of $959. It had a height of 8 feet 8 inches, was 4 feet 2 inches square and had a discharge chute 31 inches long and 20 inches wide.

In placing the mass concrete in open forms, the square-bottom dump bucket was used. In smaller forms, when it was not advisable to dump all the load at one time, the 7-cubic-yard round bucket was used. This latter bucket poured all of the small upper lifts of the dam, most of the training and gravity walls, and a large part of the concrete in the powerhouse. When forms were so located that the bucket could not be swung directly over them, or when they required very small quantities of concrete, the 3-cubic-yard side-dump bucket was used. For pouring into hoppers from which concrete was delivered to the forms by chute or for loading trucks from the cableway, either of the controllable dump buckets was satisfactory.

The first 6-cubic-yard square buckets were not completely satisfactory because of the slowness in dumping and because they would not hold 6 cubic yards of the type concrete produced. To dump these buckets required the effort of several men and often 5 or more minutes. Almost immediately experiments were begun to develop methods of improving the dumping mechanism. Air operation of the dump seemed most feasible because of its safety, and for this reason a 6-inch chamber with a 20-inch piston stroke was installed to raise the dumping bail. Air could be admitted on either of two sides of the bucket at the operator's choice—a two-way check valve in the line to the chamber directed the air to the piston. This arrangement opened the gate satisfactorily, but the bail had to be

unlatched by hand. This slowed the operation and resulted in injury to several operators' hands. A 4-inch chamber, with a piston of shorter stroke than the larger one, was placed near the 6-inch

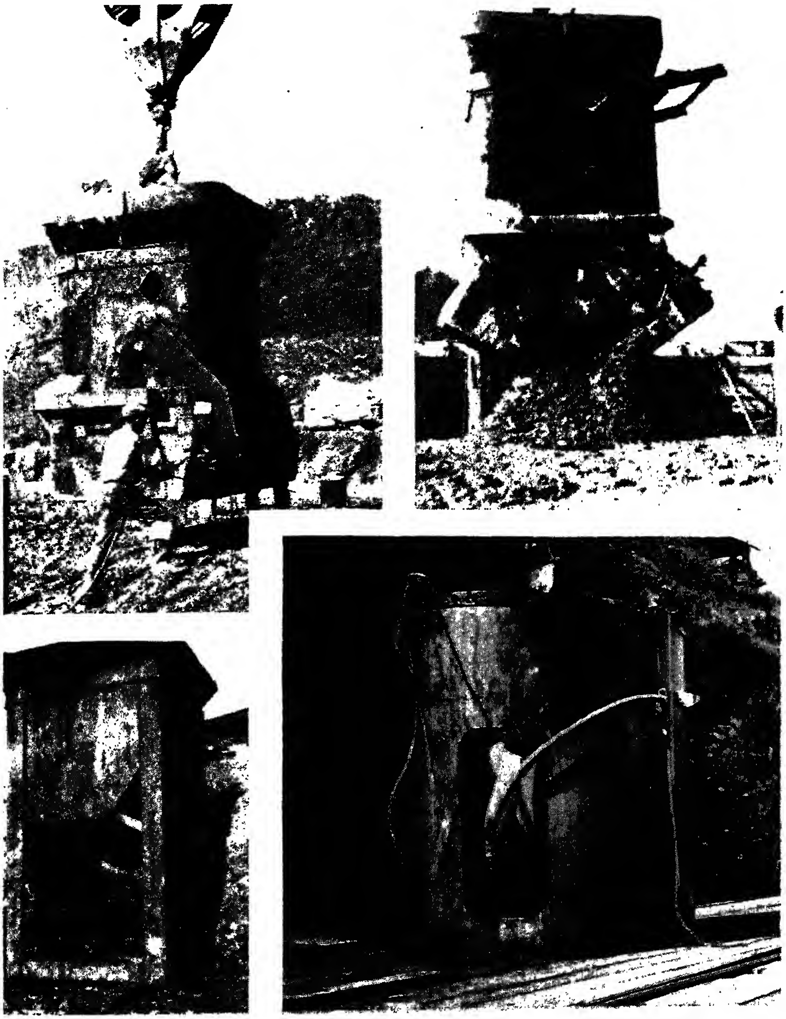

FIGURE 139.—*Concrete buckets.*

chamber and connected with the same air line. This smaller piston actuated the latch and, because it required less pressure to operate, unlatched the bail before the larger piston moved.

In the operation of these buckets, dumping required the work of only one man carrying a light air hose. A quick-opening valve and a push-fit connection was provided for attaching the hose to the receptacle on the bucket. Buckets were self-closing and, aside from occasional removal of accumulated material around the dumping mechanism and doors, required no manual attention during use.

Relatively stiff concrete was placed in the dam. When it was poured into the original square buckets, it piled up so that some was wasted on the trestle and some was left in the chute of the transfer cars. Falling material from overloaded buckets passing overhead was a menace to the workers. A 12-inch lip around the top of the bucket, inclined to the side walls of the bucket, proved too low to prevent this spilling, and an additional vertical section 8 inches high was added. The third of the square buckets required only the addition of the 8-inch section.

All buckets were provided with sturdy bases and side bumpers to prevent damage during landing on the platform of the cableway transfer track. Horizontal projections were fitted with sloping fillets to prevent accumulation of mortar or rocks which might fall and injure workmen below. The buckets were inspected regularly and were removed from service when worn or damaged parts were found.

Forms. (*See chapter 6.*)

Concrete in the dam was placed in 5-foot lifts in construction blocks generally 56 feet in width and up to 200 feet in length, de-

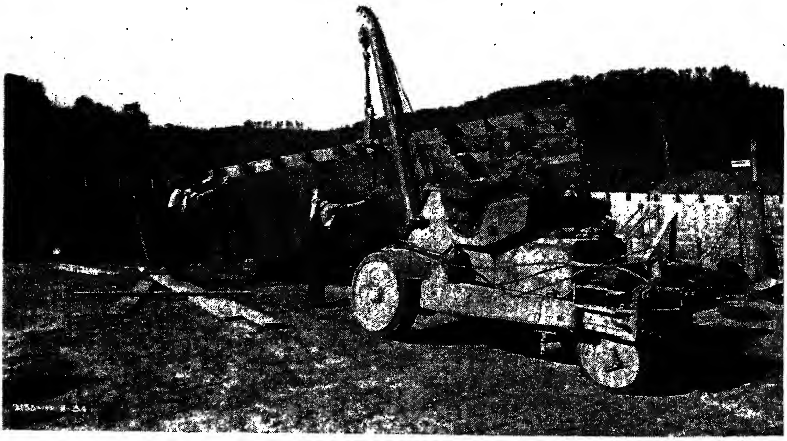

FIGURE 140.—*Krane Kar placing panel form.*

pending on elevation. Panel forms made of wood were used for plane concrete surfaces of the dam.

The forms were built in the carpenter shop complete and ready for use. The shop was built with a large platform on one end and was

so located that forms could be removed from the platform by the cableways and carried to their destination in the dam or powerhouse area. Stripping and setting of panel forms in the blocks of the dam were done by Krane Kars. Each Krane Kar unit consisted of a three-wheel, self-propelling, rubber-tired gasoline crane equipped with a 14-foot jib type boom, front wheel drive, and rear wheel steering. Boom capacity was 5,000 pounds at a 5-foot radius and 2,500 pounds at a 10-foot radius. The extreme mobility and small clearances required were features which enabled it to be used inside the forms.

Vibrators.

All concrete except that in very thin or heavily reinforced sections was consolidated by Electric Tamper & Equipment Co.'s VS-4 two-man internal vibrators operating on three-phase electric power.

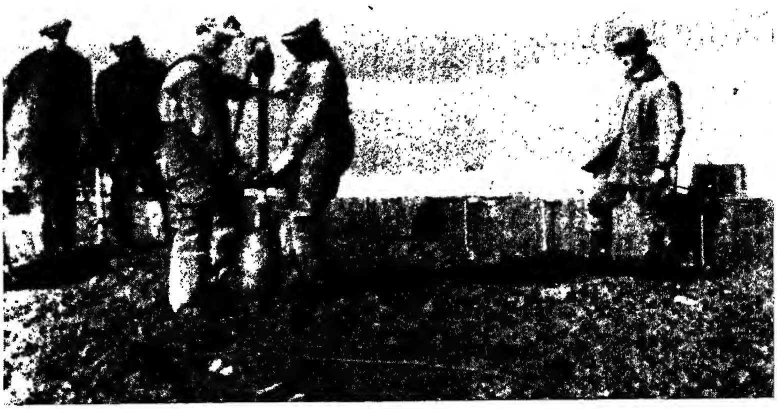

FIGURE 141.—*Vibrators for mass concrete.*

It was recognized before the vibrating equipment was purchased that 60-cycle power available would not provide a vibrating frequency desirable for placing dry concrete. Consequently, frequency changers, consisting of slip-ring motors V-belt driven by induction motors were provided to increase vibrating speed. Concrete placing started with 68-cycle power supplied to the vibrator. This frequency was increased by changing V-belt pulley sizes until 80 cycles were reached. At this frequency no-load vibrating speed was 4,800 per minute. The actual speed under load was probably about 4,500 per minute. Eighty-cycle operation was adopted in January 1935 and maintained throughout the remainder of the job. Surface vibrators or puddlers, also manufactured by the Electric Tamper & Equipment Co., were used to compact top surfaces of completed concrete lifts. Both types of vibrators operated efficiently at the highest speed, and by a very thorough inspection and maintenance, operating costs were kept comparatively low for this type of equipment.

OTHER CONSTRUCTION PLANT FACILITIES

As stated previously, the production of concrete and its placing were the primary considerations in setting up the construction plant facilities; however, other facilities were required in construction work. Equipment for excavating, grouting, and hauling; machine and carpenter shops; roads and bridges; and air, water, and power supply were all a very necessary part of this plant. Special equipment for each particular construction operation is described along with the discussion of that operation in chapter 6.

Foundation excavation, drilling, and grouting.

Spoil from all cofferdams was removed by 1¼-cubic-yard shovels and a fleet of 3-cubic-yard dump trucks. During the excavation of the first cofferdam area, this equipment was supplemented by the two 3-cubic-yard electric shovels and 12-cubic-yard dump trucks. This latter equipment was later moved to the quarry. Final clean-up of the foundation area was done by hand labor. Blast hole drilling was done by jackhammers except a part in the powerhouse section where wagon drills were used. All line drilling was done by wagon drills.

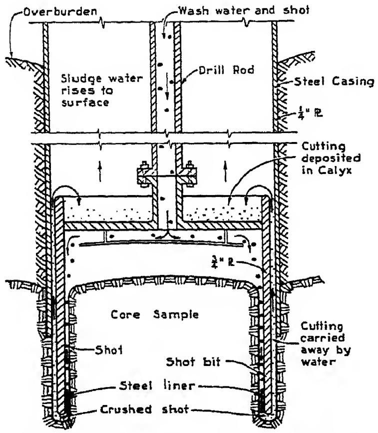

FIGURE 142.—*Calyx drill—Method of operation.*

Shallow grouting, curtain grouting, and grouting of the rim of the reservoir were the three general types of grouting performed. Three-inch and 5½-inch wagon drills, and shot core drills were used in the dam area. Diamond drills were employed for most of the rim drilling, but two churn drills were used for a small portion of this work. The diamond and churn drills worked on contract.

Four calyx core drills, three electric and one gasoline, were used for the drilling of foundation grout holes.

These drills operated 13,065 hours on 5½-inch gallery holes, 5,769 hours on 3-inch holes outside the dam, and 1,650 hours on 3-inch holes at Loyston Dike (April 1, 1935–December 31, 1935). Delays amounted to approximately 10 percent of the gross operating time.

Drilling rates per gross operating hour were: gallery holes, 1.22 feet per hour; outside holes, 1.79 feet per hour; and Loyston Dike holes, 1.63 feet per hour.

Core drilling costs for the 5½-inch holes were $3.67 per foot; for the 3-inch holes, $2.64 per foot; and for the Loyston Dike holes, $1.69 per foot. A summary of quantities and costs for this work is shown in table 75.

Each operating crew consisted of one operator at $1 per hour and one helper at $0.60 per hour. In driving casings, one additional helper per shift was added to each crew.

TABLE 75.—*Summary of drilling quantities and costs*

Type	Size (inches)	Overburden (linear feet)	Rock (linear feet)	Total (linear feet)	Total unit cost
Wagon	2½	0	149,308	149,308	$0.408
Shot core	3	4,532	15,363	19,895	2.634
Shot core	5½	0	32,248	32,248	3.660
Shot core	36	0	651	651	------
Diamond core [1]	2 1/16	1,935	7,721	9,656	2.25
Diamond core [1]	2 5/16	4,054	26,344	30,398	2.75
Well [1]	6¼	844	3,713	4,557	2.40

Grouting dam foundation:
- Cement .... 202,770 cubic feet
- Rock flour .... 284 cubic feet
- Total .... 203,054 cubic feet at $1.521 per cubic foot

Grouting reservoir rim:
- Cement .... 140,380 cubic feet
- Rock flour .... 115,803 cubic feet
- Sand .... 1,553 cubic feet
- Total .... 257,736 cubic feet at $0.535 per cubic foot

[1] Contracted.
[2] Includes cost of pipe and fittings at $0.05 per cubic foot of grout injected and cost of washing seams at $0.21 per cubic foot of grout injected.

The equipment used for mixing the grout consisted of a two-compartment, open-top mechanically-agitated mixer that discharged by gravity into the suction of a 7- by 5- by 10-inch, air-driven, duplex, reciprocating pump. A close-quarter, air-driven drill motor, mounted on a bracket at the end of the mixing tank, was used to drive the agitating blades. The pump and mixer were mounted together as a portable, three-wheeled unit that was easily pulled to any part of the job. The mixer was divided into two compartments so that an uninterrupted flow to the pump might be obtained by mixing in one compartment while discharging from the other. A valve in the branch of the pump suction from each compartment permitted the pump operator to maintain a continuous flow by changing the valves at the proper time. The open-top mixer possessed the advantage of being easily cleaned, as all parts of the interior were accessible. Repairs were facilitated for the same reason.

The maintenance of the mixers was not an important item, as wear of the shaft and the stuffing boxes where the shaft passed through the end walls and partition was not excessive if reasonable care was exercised in keeping the glands packed. On the pumps, the demand for the repair and replacement of valves, piston packing, piston rods, and cylinder liners was constant and was considered as part of the operation. The fluid pistons were of cast iron with rubber packing, and the cylinders were made with removable steel liners. The rate of wear of the rubber piston packing against the steel liners was found to depend largely upon the pressure at which the pump was operating, and the wear was about equal on the steel and on the rubber. The life of the cast iron valve seats was also found to depend to a great extent upon the pumping pressure. These seat rings were removed in the field and were refaced in the shop. Valve discs of medium rubber were found to be far superior to the balata discs furnished as original equipment as, by reason of

their greater ability to conform to slight irregularities in the seat, they were more effective in preventing leakage. The abrasive action of grout was so great that, once a small leak started, only a short while was required for considerable damage to result. To a lesser extent than on the parts just mentioned, piston rods were also subject to wear. Piston packing and valve replacements were usually made in the field though it was necessary to have cylinder liners removed and replaced in the machine shop. Four complete grouting units were kept in service, and one spare fluid end for the pumps was held in good repair to facilitate replacements.

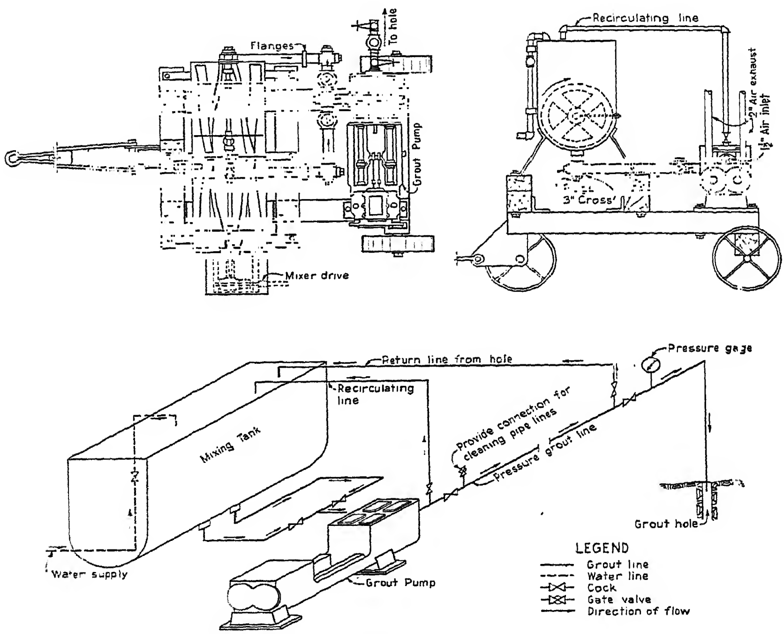

Figure 143.—*Grouting machi*

The greatest source of trouble with regard to wear on the pumps was found to lie in the steel particles that were contained in the cement. These varying in size from very small particles to fragments weighing several ounces blocked valves, scored cylinders, and, in general, increased the wear on the pump appreciably. They were found consistently in the three brands of cement used, and complaints to the manufacturers elicited promises and regrets that did not help to reduce the proportion of steel particles. At various times during the grouting operations, screens were installed for the purpose of removing foreign matter from the fluid grout. However, they proved such a constant source of trouble and delay that it became desirable to abandon their use. Dry screening of the cement through a screen small enough to stop all harmful particles would have been

a rather expensive operation. A quickly replaceable screen located between mixer and pump would prove most satisfactory as hardened particles of cement would be removed with the steel.

Shops and buildings.

The machine shop, carpenter shop, field engineering office, and the majority of shop buildings were located on the east bank below the dam site and near the powerhouse area. On top of the hill west of the dam were the main office and main warehouse. The location of the plant buildings is shown on the lay-out sketch, figure 104.

Transportation facilities.

Workers were transported at no charge from the town of Norris to the dam, about 5 miles, by 5-ton trucks with combination freight and passenger bodies operating on regular schedules. The total cost of operating these passenger buses, as determined by a special study, was $0.176 per mile or $0.007 per passenger mile, with a passenger trip consisting of 5 miles. Freight was transported from Coal Creek and Knoxville by trucks or by trailers. General miscellaneous hauling about the job was cared for by two 8-cubic-yard and one 1½-ton dump trucks. Gasoline was distributed by a 1½-ton truck that had a 400-gallon tank mounted upon it. Operation cost for a period of maximum activity determined by a special study for the utility trucks is shown below:

Type of truck	Total cost per net operating hour	Total cost per mile
2½-ton stake body	[1] $1.684	[1] $0.147
5-ton combination body	[1] 1.685	
8-cubic-yard dump	[2] 2.820	
1½-cubic-yard dump	[2] 1.460	
Gasoline	[2] 1.460	

[1] Cost to Dec. 20, 1935. [2] Cost to Mar. 20, 1936.

Any work requiring the use of a tractor was cared for by three Allis-Chalmers tractors, two model L, one model K, and a Caterpillar "35" tractor. From special studies, operation costs per net operating hour for the tractors were determined to be approximately as follows:

	Allis-Chalmers model L	Allis-Chalmers model K	Caterpillar "35"
Size horsepower (drawbar)	79	50	--
Operating cost (net operating hours)	$3.63	$3.05	$2.91

Reinforcing steel handling.

Reinforcing steel was stored below the dam on the east bank of the river. All bends were made near the storage yard by a horizontal electric bender, a circular bender, or a hand bender. Normally, the steel was transported from the storage yard to the dam by wagons, but at times trucks were used. At the dam site it was lifted to its place in the dam by the cableways.

At the steel yard, material was unloaded in separate piles according to size, length, and type. These piles were so arranged that steel was stacked in layers and separated by 1-inch wooden strips. At one end of the pile all bars were made even. A wooden stake driven at the even end of each pile recorded the size, length, and type of bar. This method made it easier to take physical stock of the yard, made possible the use of an acetylene torch to cut bars without burning those underneath, and required only the handling of the length of bar desired without rehandling the left-over portions.

Unloading equipment.

For unloading heavy equipment at the railroad terminal at Coal Creek, a locomotive crane was provided. For handling heavier equipment than could be handled by the crane, a large A-frame with a 100-horsepower Lidgerwood hoist was used.

The crane was an Orton-Steinbrener, standard-gage, locomotive crane with a 50-foot boom. At a radius of 12 feet, its capacity was 36,000 pounds, and at 40 feet its capacity was 7,800 pounds. Power was obtained from an 80-horsepower steam boiler for traveling, hoisting, and sluing operations. The crane was secured from Nitrate Plant No. 2 at Muscle Shoals at a cost of $5,000. Under normal conditions the operating crew consisted of one operator at $1.50 and one fireman at $0.60 per hour. Additional men made up the unloading crews, the size of which varied with the type and quantity of material received. The crane operated a total of 2,812 hours at a cost of approximately $3.85 per hour.

Riprap placing.

The clay fill in the east end of the dam required the placing of 32,200 square feet of riprap for protection against wave action. This riprap was handled by a job-made skid derrick on which the total operation and depreciation charges amounted to $1,637.56. A total of 6,856 tons of derrick-size rock was handled for $0.24 per ton or $0.051 per square foot.

Air distribution.

Compressed air was provided by one Bury and three Sullivan compressors with a combined capacity of approximately 7,600 cubic feet per minute and distributed to the job through two 6-inch mains and auxiliary lines. One line was located on each side of the river. The compressor house was located on the west bank of the river downstream from the construction bridge. Four auxiliary receivers located near the point where most air was used and a receiver at the compressor house provided sufficient storage.

Water supply.

The estimated requirement for raw water was 118,000 gallons per hour and for fresh water, 3,750 gallons per hour. Raw water was pumped from the river below the dam site to the two 20,000-gallon storage tanks above the west abutment by two 8-inch centrifugal pumps operating in tandem. Two 5- by 4-inch pumps were maintained as spares. The fresh water supply came from the same source as the supply for the town of Norris and was stored in two 20,000-gallon tanks near the raw water storage.

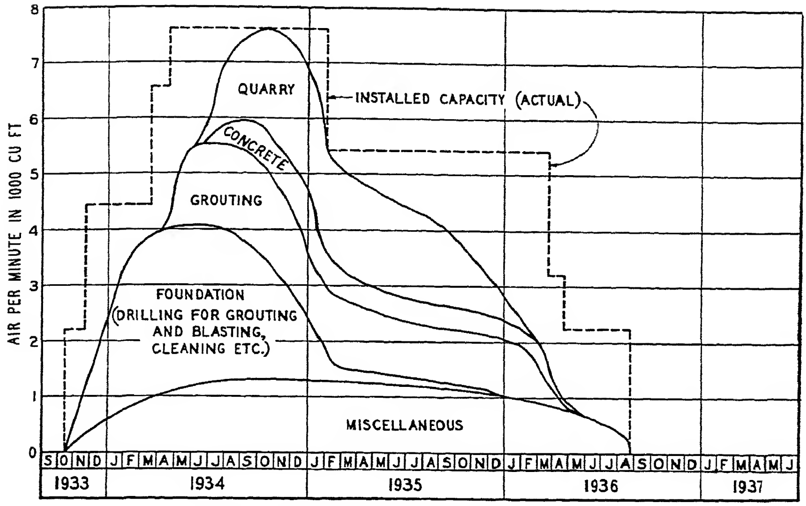

FIGURE 144.—*Estimated compressed air use.*

Electrical distribution.

A substation, located on the hill above the west abutment, was served by a 66,000-volt supply line, three 2,300-volt distribution lines carried the power to the different parts of the job. Transformers were located at convenient points to step down the current and the desired voltage for the equipment.

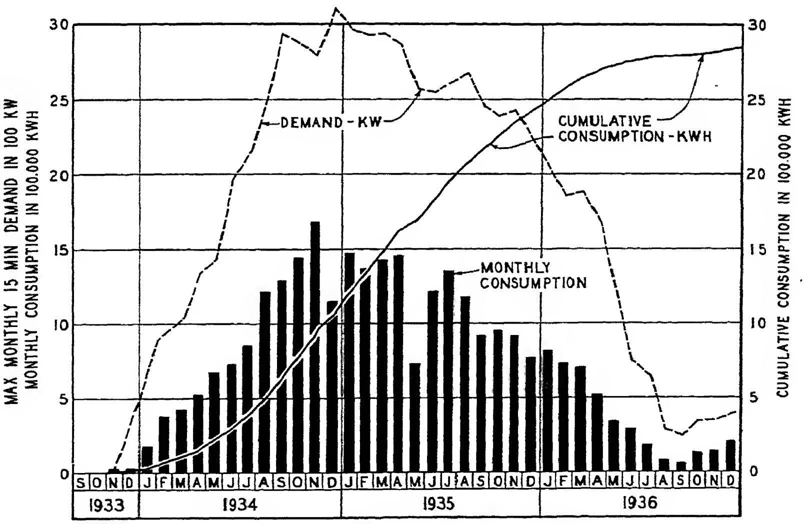

FIGURE 145.—*Electric power consumption.*

Roads and bridges in the construction area.

Transportation of materials, equipment, and men within the construction area principally by trucks, tractors, trailers, and automobiles necessitated the construction of roads in that area. An important factor which had to be taken into account in laying out and maintaining the roads in the construction area was the fact that the freeway from Coal Creek to Norris on the west side of the river and from Norris Dam to the town of Norris on the east side offered an improved and shorter highway between United States Highway 25W at Coal Creek and Knoxville. As a matter of policy, the freeway was open to the public as soon as it was completed to the construction area, early in 1934; and the temporary construction road bridge linking the two portions of the permanent freeway assumed the character of a public highway, as well as a construction facility, until the permanent roadway across the dam was ready for use in September, 1936.

A construction bridge located 1,100 feet downstream from the axis of the dam was about 470 feet long and was provided with a 14-foot roadway and a 4-foot 2-inch walkway on the upstream side. This bridge consisted of steel bents on concrete piers, with steel I-beam girders and a wooden superstructure. The bridge was ample for normal requirements. In order to avoid the necessity of partially dismantling the larger shovels crossing the bridge, and to take care of the heavy permanent equipment—the heaviest items were 161-kilovolt transformers weighing 111,000 pounds each—the bridge was reinforced by placing timber bents of used 10- by 10-inch and 12- by 12-inch timber at the midpoint of each span.

Construction of the bridge was started October 13, 1933, and the first motor traffic crossed on December 2. It was kept in service until August 1936 when it was dismantled. The bents and girders of the bridge were shipped immediately to Hiwassee Dam and it was erected there to serve in the same capacity.

RIVER DIVERSION

The general scheme of river diversion and of dam closure was established through studies and alternate plans developed by both the United States Bureau of Reclamation and the Authority's engineering organization. During dam construction operations three principal cofferdams were used for river diversion, unwatering successively the east side, the west side, and the middle of the river. Inside these cofferdams excavation, foundation treatment, and concreting were carried on in the dry. Five monoliths or blocks of the dam were left low for river diversion, three of which were concreted during low water by normal forming methods. The remaining two were alternately closed and concreted in 5-foot lifts by means of a special steel closure gate until the entire flow of the river was diverted through the permanent conduit outlets.

PROPOSED METHOD OF DIVERSION

The probable river flow, the schedule of concreting operations, the hazard incident to several proposed closure methods, the economy of the general construction program, and the economy of the actual cofferdam and closure operation were all principal factors involved in the consideration of the various schemes of river diversion. An additional, and in this case a dominant, factor was the necessity for starting work as soon as possible in order to provide employment on construction work without delay. This was actually the determining factor in starting stream diversion at the beginning of a high-flow period rather than at the beginning of a low-flow period. Thus started, it was necessary to build two high cofferdams capable of withstanding high water stages during winter months and one low cofferdam for low-flow summer months. Under other circumstances, where a starting date could be chosen, two cofferdams, one high and one low, would probably have been enough.

At the site, the river channel is approximately 300 feet across, with no cover or overburden on the hard dolomite bedrock. Flow varies from a minimum of 400 cubic feet per second to a flood peak of 70,000 cubic feet per second. The vertical rise for this variation in flow is about 40 feet. During the high-water period, which occurs between November 1 and July 1, maximum floods range between 45,000 cubic feet per second and 70,000 cubic feet per second; and during the low-water period, which occurs between July 1 and November 1, maximum discharges seldom exceed 25,000 cubic feet per second and the average is much less. The highest and second highest maximum mean daily discharges (in thousands of cubic feet per second) by months for a period of 35 years (1898–1932) are given in table 76.

TABLE 76.—*Highest and second highest maximum mean daily discharges by months, 1898 to 1932*

	January	February	March	April	May	June	July	August	September	October	November	December
Highest	[1] 69	57	70	54	45	45	20	29	12	26	54	60
Next highest	54	55	56	40	43	41	20	27	12	16	40	51

[1] Considered 70,000 cubic feet per second.

Frequency of occurrence of floods: 70,000 cubic feet per second, 1 in 17.5 years; 60,000 cubic feet per second, 1 in 11.7 years; 54,000 cubic feet per second, 1 in 3.9 years.

For year-round purposes based on the maximum mean daily discharges the adequacy of various diversion capacities is given in table 77.

TABLE 77.—*Adequacy of various diversion capacities*

Diversion capacity (cubic feet per second)	Years when capacity is ample to pass peak flood	Years when capacity is inadequate to pass peak flood
30, 000	9	26
40, 000	13	22
50, 000	21	14
60, 000	29	6
70, 000	33	2

Initial study.

The initial study for stream diversion and dam closure submitted to the Authority by the Bureau of Reclamation in September 1933 included an analysis of hydrological data and fixed the desirable diversion capacity at 60,000 cubic feet per second.

The first scheme called for a steel sheet pile cofferdam on the east side of the river. Some excavation was to be done in the dam area on the west bank to increase diversion capacity. For subsequent diversion five blocks were to be left low and six tunnels were to be provided in the section of the dam constructed within cofferdam No. 1. A second steel sheet pile cofferdam was to be driven across midchannel to the west bank as soon as concrete in the first cofferdam was sufficiently high. Provision for later diversion was to be made by leaving three blocks low and providing four tunnels in the section of the dam constructed within this cofferdam. The eight low blocks of first- and second-stage construction and two of the tunnels would then be concreted during the low-water season of 1935. The remaining eight tunnels would then handle up to 60,000 cubic feet per second after all concrete had reached a height to permit the water in the reservoir to raise to elevation 900. The tunnels were to be closed during the summer of 1936.

The second scheme was much the same as the first, except that the first cofferdam was proposed on the west side of the river. This permitted work in the powerhouse area to be started at an earlier date than in the first scheme.

The third alternate was drawn up to show what could be done if the full summer were available for starting work. This scheme in-

volved the building of one low-water steel sheet pile cofferdam capable of withstanding 25,000 cubic feet per second, and two additional cofferdams, one of steel sheet piling and one of timber cribs, high enough to withstand a flow of 60,000 cubic feet per second. Final closure was similar to the first two schemes. This scheme was eliminated since it did not fit in with a rapid construction schedule.

Changes were made in details of all schemes. Timber cofferdams were substituted for sheet steel piling of cellular construction for reasons of economy since timber was available in large quantities at low cost. Other changes, including a revision in location of construction joints between blocks and a modification in the design of the dam, were also taken into account. With these revisions a more detailed study of the first two schemes was made.

Revised scheme No. 1.

The first plan as revised required the use of two timber crib cofferdams, the first built from the east bank and the second from the west bank. Diversion was to be over low blocks of the dam and through conduits which were to be closed later. In general, the scheme was similar to those first submitted. The first or east cofferdam was to unwater a portion of the dam and apron and a portion of the tailrace area, and was to be high enough to allow a flow of 60,000 cubic feet per second to pass through the restricted channel. Within the first cofferdam area all concrete was to be placed to a minimum elevation of 862, except in three blocks which were to be left at elevations 843.5, 835.0, and 825.0. In addition to the low blocks, four 10- by 15-foot conduits were to be provided. These low blocks and conduits provided for a stream flow of 60,000 cubic feet per second after the first cofferdam was abandoned, and the second cofferdam on the west side of the river built and placed in service. Within the second cofferdam, all concrete except in two blocks was to be placed to a minimum elevation of 862. Four 10- by 15-foot conduits were to be provided in addition to the two blocks which were to be left at elevation 843.5. When this work was completed the second cofferdam could be dismantled. The entire dam could then be raised uniformly except for three blocks in the spillway section which were to be kept at least 15 feet lower than any other until the lowest one approached elevation 880. When the conduits alone could discharge 60,000 cubic feet per second, concreting could proceed without further limitations. The final closure of the conduits was scheduled for the low-water season of 1936.

Revised scheme No. 2.

Much the same as in revised scheme No. 1, this scheme required the use of two timber crib cofferdams. The first or west cofferdam was to unwater the entire western area of the dam. A diversion channel to help provide for a flow of 60,000 cubic feet per second was located in the area of the future dam, powerhouse, and tailrace on the east bank. During construction within the first cofferdam three blocks were to be left low and eight 10- by 15-foot conduits were to be provided for subsequent diversion. When construction within the first cofferdam was completed, it was to be dismantled and the second

cofferdam built to unwater the east side of the river. A flow up to 60,000 cubic feet per second could be passed through the conduits and over the low blocks of stage 1 construction. Concreting could then be carried on within the area of the second cofferdam without the necessity of providing either conduits or low blocks for subsequent diversion. As soon as concrete in this area was brought up to elevation 860, the second cofferdam could be dismantled and the entire dam could be concreted in the same manner as in revised scheme No. 1.

ADOPTED SCHEME OF DIVERSION

In the adopted scheme of diversion the first cofferdam was to be built from the east bank, mainly because the maximum amount of work could be started at the earliest date and because the powerhouse

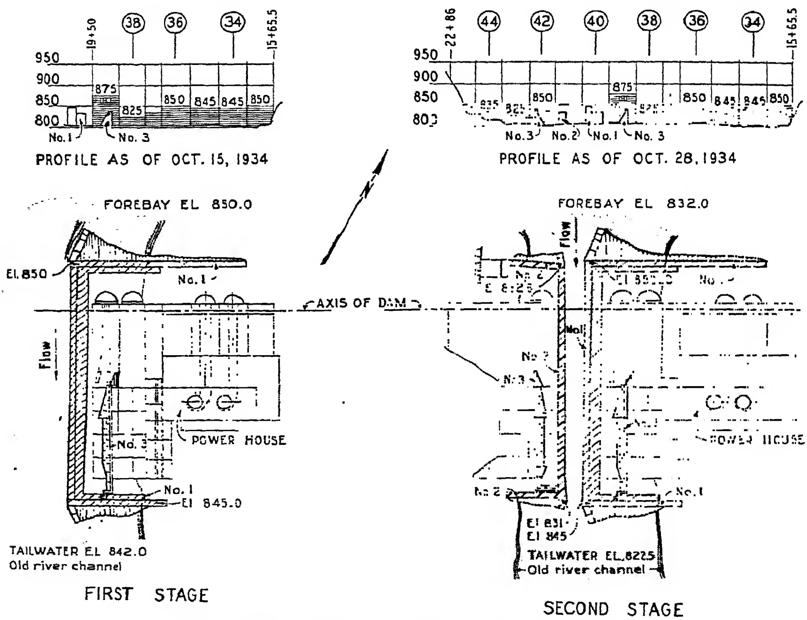

Figure 146.—*River diversion schedule for first and second stages.*

area would be continuously available for construction. The use of diversion conduits was abandoned due to the hazard of possible clogging by floating debris carried by the river in flood stage. The alternate method which was adopted was to leave several blocks in the spillway section as near the river bed elevation as possible. These blocks were to be closed later in the life of the job by alternately closing off and concreting 5-foot lifts in successive blocks. This work was to be done during the low-water period until the water level of the reservoir reached the intake for the permanent

spillway outlet conduits, after which all diversion was to be through these conduits.

Additional studies indicated that construction within the area of the first cofferdam on the east bank would not have progressed far enough to permit river diversion over the low blocks constructed inside of it until the early winter of 1934–35. However, a low cofferdam on the west bank could be built and used during the low-water season of late summer and fall of 1934 to allow the entire west abutment to be constructed. This low cofferdam was designed to be overtopped at a flow between 3,500 and 4,000 cubic feet per second, and it reduced slightly the flow which the first cofferdam could withstand. If two blocks of the dam in the area of this cofferdam were left low for diversion purposes, the remaining area in the middle of the river could then be cofferdammed during the remainder of

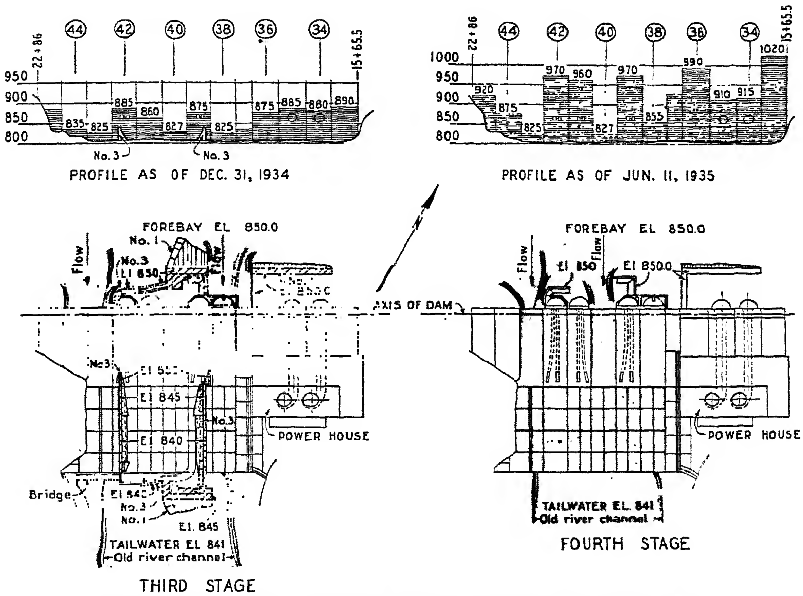

Figure 147.—*River diversion schedule for third and fourth stages.*

the low-water season and before diversion could be started through the first or east cofferdam. This would occur approximately at the beginning of the high-water season. Construction of the third cofferdam would be less hazardous if performed during the low-water period, and some economies were anticipated in cofferdam construction.

The advantages of this scheme were that it provided for a more uniform and rapid concreting schedule, which resulted in substantial economies in concreting costs, and it reduced the hazard in closures by eliminating possibilities of clogging bypass facilities. It reduced

cofferdam costs without causing additional costs by generally accelerating the construction schedule. It was conservatively estimated that approximately $83,000 was saved by the use of this scheme over the first schemes proposed. This estimated saving is summarized in table 78.

TABLE 78.—*Summary of estimated saving on revised cofferdam scheme*

Material	Quantity		Cost
Reinforing steel	223 tons at	$60.00	$13,380
Forms	90,000 square feet at	.60	54,000
Bulkhead gates	8 at	1,000.00	8,000
Concreting diversion tubes	11,000 cubic yards at	1.00	11,000
Total			86,380
Closure gate and handling—deduct			3,380
Net saving			83,000

River diversion and closure actually took place in four stages.

Cofferdam No. 1.

Cofferdam No. 1, or the east cofferdam, enclosed an area of 3.7 acres. Both the upstream and downsteam arms were constructed of timber cribs. The water seals for these cribs were made by placing a clay fill on the outside face of the crib. The outstream arm joining the upstream and downstream arm was built almost normal to the dam axis. It consisted of two lines of sawed timber cribs 14 feet wide and separated by a clay-sealed chamber 8 feet wide. The cribs are tied together with three 1-inch round tie rods spaced on 14-foot centers. A river deflection crib slanting toward the east bank was located on the upstream and downstream ends of the outstream arm and served to deflect the water and preserve the clay toe fill outside of the upstream and downstream arms.

All cribs were built of 8- by 8-inch sawed timbers and sheathed with 1-inch lagging. All fill for the cribs was of rock and clay obtained from excavation in the dam and powerhouse area. The upstream and downstream arms were paved solid with 8- by 8-inch timbers and used as a roadway.

The cofferdam was endangered by floods only once, when on March 5, 1934, a flood of 50,000 cubic feet per second occurred. Possibilities of a higher discharge made it advisable to allow the cofferdam area to fill with water to equalize the pressure on the walls of the cofferdam and to minimize damage in case of overtopping. The cofferdam, however, was not overtopped. With this exception the cofferdam continued in service until December 13, 1934, when the river was diverted over blocks Nos. 37 and 38. Some inflow of water under the cribs was experienced during the earlier days of service, but it was stopped by drilling in the rock of the river bed and by grouting with neat cement mortar.

Cofferdam No. 1 remained intact during the period of service of the second cofferdam, and dismantling was not started until the third or middle cofferdam was completed and unwatered. Dismantling

was started October 11, 1934, on that part of the outstream arm of the first cofferdam which was inside of the middle cofferdam. A part of this outstream arm was left to serve as a temporary access ramp to the river bottom in the middle cofferdam. When a trestle ramp was completed on November 15, 1934, the balance of this portion of the outstream arm was removed. The upstream and downstream

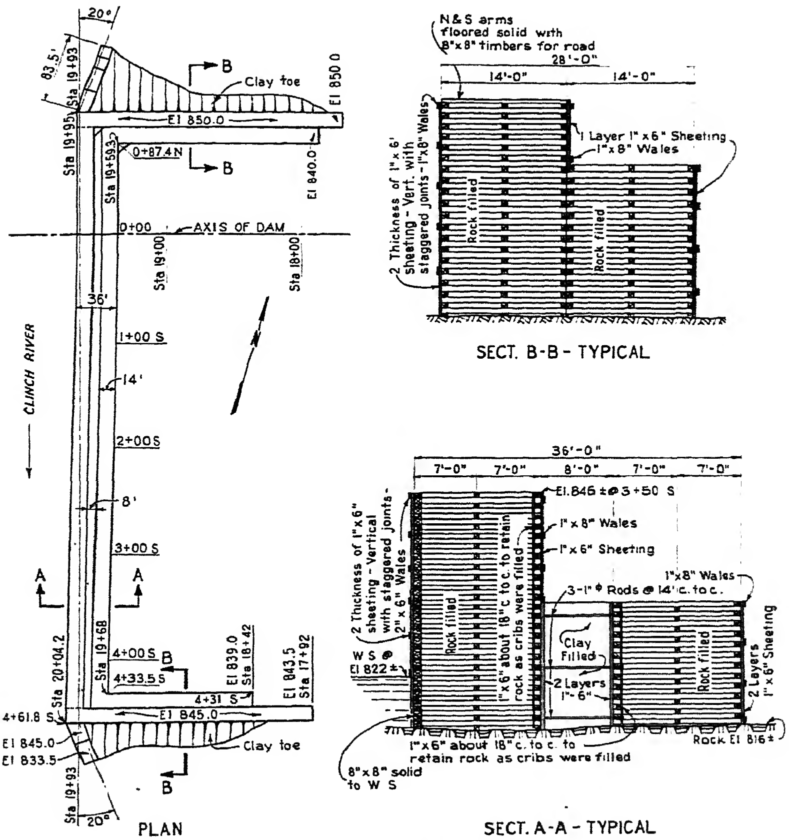

FIGURE 148.—*Cofferdam No. 1.*

arms of cofferdam No. 1 were not touched until early December 1934 when the entire downstream arm and the major portion of the upstream arm were removed to permit partial diversion of the river over blocks Nos. 37 and 38. The cribs which were originally used for the first cofferdam and later used as a part of the middle or third cofferdam were not removed until February 1935 when river flow was diverted through the section of the dam constructed within the

third cofferdam. Construction quantities and cost for the first cofferdam are given in table 79.

TABLE 79.—*Quantities and costs—Cofferdam No. 1*

Description	Quantity	Total cost
Crib work, 8- by 8-inch timber	978, 600 f.b.m.	
Crib work, 1- by 6-inch sheathing	173, 200 f.b.m.	
Total crib work	1, 151, 800 f.b.m.	$76, 177. 76
Crib fill and seal blanket	35, 500 cu. yd.	14, 835. 17
Unwatering and maintaining water level and structure 320 days		43, 157. 19
Protection from floods		7, 332. 56
Dismantling and removing		27, 854. 31
Job prorata		3, 490. 54
Total		172, 847. 53

Cofferdam No. 2.

Cofferdam No. 2, or the west cofferdam, enclosed an area of approximately 1.7 acres. Both of the upstream and downstream arms of this cofferdam were constructed of 8- by 8-inch sawed timber crib with 1-inch sheathing, and both were filled with clay and rock from the dam foundation excavation and sealed against leakage by the clay sealing blanket on the river side of each arm. The outstream arm was of the Ohio River box type consisting of two timber walls 12 feet apart, each made up of two layers of 1- by 6-inch vertical sheathing, supported by 8-inch square wales on the outside and tied across by ¾- and ⅞-inch round steel rods placed on 5-foot centers. The walls were braced with 2- by 8-inch timbers both laterally and transversely. The space between the walls was filled with clay, and a 6-inch layer of lean concrete was placed on top of the clay to protect it from erosion in case of overtopping. Deflection arms were built at the upstream and downstream end of the outstream arm to guide the water and protect the clay toe seal against washing.

The first unwatering of the second cofferdam was begun on August 9, 1934, and completed on August 11, 1934. The concreting required in this cofferdam area was carried above danger line from floods before October 26, 1934, leaving blocks 43 and 44 low for subsequent diversion.

TABLE 80.—*Quantities and costs—Cofferdam No. 2*

Description	Quantity	Total cost
Excavation (labor only)		$679. 73
Crib work	187, 700 f.b.m.	18, 422. 28
Crib fill and seal blanket	9, 820 cu. yd.	4, 870. 68
Concrete (6-inch layer over top)	85. 4 cu. yd.	968. 05
Unwatering and maintaining water level		14, 977. 40
Flood protection		1, 622. 06
Dismantling		4, 476. 66
Job prorata		948. 43
Total		46, 965. 29

In order to create as small a flood hazard as possible to the east cofferdam, the west cofferdam was held to a minimum height, and it was expected that small rises might flood it. Rises in the river on

August 14, 1934, and August 25, 1934, threatened to overtop the wall, and, therefore, the cofferdam was flooded on both days. Only 5 days were lost on these occasions out of a total of 76 days of service. Actually only once, on August 25, at a discharge of about 3,500 cubic feet per second, did the river rise as high as the top of the cofferdam.

Some trouble was experienced from leakage under the downstream

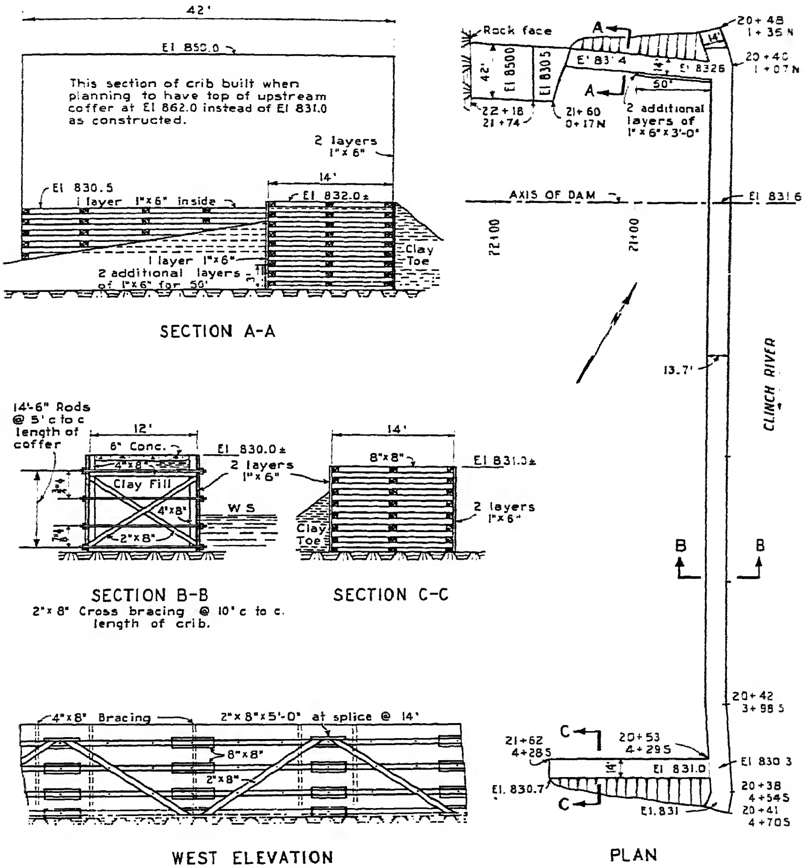

FIGURE 149.—*Cofferdam No. 2.*

end of the outstream arm. An auxiliary cofferdam enclosing the area in which the most serious inflow occurred was found advisable. Finally, the cofferdam was flooded to equalize pressure while the area affected was grouted. This flooding took 5 days, in addition to the 5 days previously mentioned, during which the cofferdam was flooded due to high water.

Quantities and costs for the second cofferdam are given in table 80.

Cofferdam No. 3.

Cofferdam No. 3, or the middle cofferdam, surrounded and unwatered the area in the middle of the river which had not been unwatered by the previously constructed first or second cofferdams. Blocks 39 and 42 of the dam, already constructed well above danger from high water, were utilized for cofferdamming the east and west sides of the area for that portion of required distance covered by their thickness. Upstream and downstream from these blocks gravity

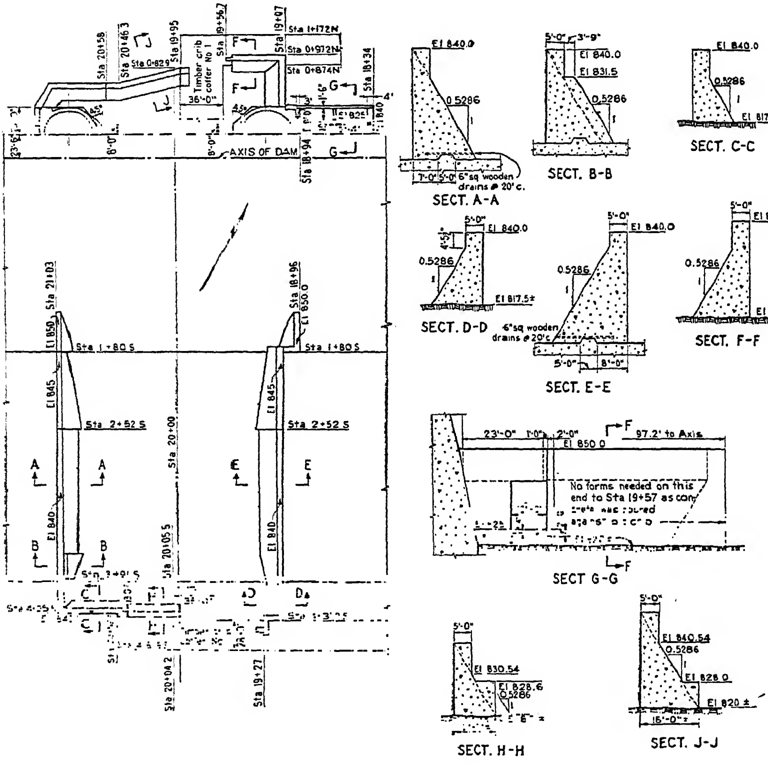

FIGURE 150.—*Cofferdam No. 3.*

walls of lean concrete formed the remainder of the longitudinal area. The downstream temporary wall rested on the previously completed spillway face and apron. Upstream and downstream arms, also of lean concrete and with gravity sections, join the longitudinal arms. At the northeast and southeast corners, part of the cribs originally built for the first cofferdam were again used as a part of the third cofferdam.

Concrete walls instead of timber cribs were resorted to, both because of the limited space, and because of the low increment cost for

concrete since the construction plant for concrete production was already in operation.

The entire east and west arms both upstream and downstream from the dam were built in the dry inside the east and west cofferdams, the work being performed as soon as adjoining permanent structures were completed. The upstream and downstream arms necessarily awaited the dismantling of the west cofferdam, or cofferdam No. 2, and the closing off of the channel in which they were to lie.

Since this cofferdam was in the middle of the river with diversion channels on each side, a construction bridge was built between the west bank of the river and the downstream end of the west arm of the cofferdam to provide access for trucks, shovels, and other equipment needed within the cofferdam.

The third stage of river diversion was started October 27, 1934, with the placing of a fill across the old channel between the east and west cofferdams. This diverted the water over blocks 43 and 44. The diversion was completed on October 28, when another temporary dike was placed across the downstream end of the channel; and unwatering in the middle cofferdam area was begun immediately. The area was kept unwatered for 106 days, until February 10, 1935, by which date it was finally allowed to fill with water. Enough of the upstream and downstream arms were dismantled by February 10, 1935, to allow water to pass over block 40 which had been left low for the purpose.

Cofferdam No. 3 was designed to withstand a river flow of 50,000 cubic feet per second, but no floods of that magnitude occurred during this period of service. No difficulty arose from either high water or leaking, except for 2 days in January when floodwaters topped the partially dismantled upstream arm.

TABLE 81.—*Quantities and costs—Cofferdam No. 3*

Description:	Total cost
Crib work	$4,236.63
Crib fill	1,519.49
Clay blanket and seal	815.69
Concrete, 9511.2 cubic yards	70,180.24
Unwatering and maintaining water level	9,820.20
Crib and crib fill removal	1,941.21
Dismantling concrete cofferdam [1]	15,668.38
Job prorata	2,147.42
Total	106,329.26
Bridge and ramp to cofferdam No. 3	13,208.33
Total cofferdam No. 3 and access bridge and ramp	119,537.59

[1] Whole cofferdam was dismantled by blasting. Only that portion downstream was removed from river bed.

At first only that portion of the upstream arm in front of diversion block No. 40 was dismantled. The remainder of the upstream arm and the entire portion of the east and west arms upstream from the dam were left in place. The east and west arms downstream from the dam which rested on the spillway face and on the apron were not removed until the spring of 1936, after the closing of the spillway outlet conduit gates. In removing the concrete cofferdam structures resting on the spillway apron, precautions were taken not to damage the concrete of the apron in any way. Clean-up of

the bottom section was carried on to a large extent with a depth of 6 feet of water over the spillway apron to prevent any possibility of damage from hydrostatic uplift.

Quantities and cost for the third cofferdam are given in table 81.

Unwatering miscellaneous areas.

A cofferdam needed for the unwatering of the penstock trashrack structures area was started about November 15, and completed about December 15, 1934. To unwater this area a timber cofferdam was built from block 36 upstream to the original ground of the east bank. The cofferdam was unwatered on February 8, 1935. At various times the area was inundated by floods, but no serious difficulties were encountered. By early June construction of the penstock intake trashrack had reached a point where the use of the cofferdam could be discontinued. The cost of this operation as of September 19, 1938, was $5,036.92.

A summary of costs for work necessary for protection from floods after abandoning cofferdam No. 3 is given below:

Description:	*Total cost*
Timber work (bulkhead over penstocks) 37,450 f.b.m.	$12,752.43
Pumping	11,960.25
Dumped riprap	1,597.60
Job pro rata	542.26
Total	26,852.54

In addition to the cofferdams previously discussed there was constructed a secondary cofferdam known as cofferdam No. 4 between the downstream end of the east training wall and the east bank of the river which, together with the east training wall and the main body of the dam, enclosed the powerhouse and tailrace areas. This cofferdam was built in the dry inside the east cofferdam area and was completed late in November 1934; however, the area was not unwatered until March 1, 1935, when construction in the powerhouse area was begun. This cofferdam was kept unwatered for 374 days, or until March 9, 1936. All drainage in the galleries of the dam, leakage of the head gates, and other leakage were diverted to this area, therefore more pumping over a longer period of time was required than would otherwise have been needed. The quantities and costs involved in this work are given in the following table:

	Quantity (cu. yd.)	Total cost
Concrete	250.4	$1,627.85
Unwatering and maintaining water level		33,381.19
Flood control		538.79
Job pro rata charges		732.66
Total		36,280.49

Other high water protection.

After February 10, 1935, when the first, second, and third cofferdams, as well as the auxiliary cofferdams protecting the penstock intake trashrack areas were no longer needed, certain construction work still remained to be protected from unusual floods. In view

of the restriction of the flow of the river through the low diversion blocks of the dam, there was the danger that floods might raise the upstream stage to such a point as to interfere with work on the penstock head gates, seats, and guides, and to cause flow through the penstocks into the powerhouse area. Accordingly, temporary stop logs were installed in the penstock intake trashracks up to elevation 880 as soon as these were built. This protected the head gate installation area against flow as high as 7,650 cubic feet per second.

As a further precaution, wooden bulkheads were installed in each of the penstock intakes downstream from the dam and just upstream from the steel penstock linings. These were designed to withstand a head of 100 feet. Thus, even if the stop logs upstream from the head gates were over-topped, temporarily stopping work on the gates, there would be no danger to the powerhouse area until an upstream stage of 952 was reached.

The cost of these operations and of other minor work incidental to providing flood protection subsequent to abandoning cofferdams was $25,921 to April 30, 1937.

Closure operation.

The closure phase of river diversion began with the entire flow of the river diverted over five blocks of the dam in the spillway and west abutment.

Although specifications for concreting in the dam required that each 5-foot lift slope 5 percent upward in the downstream direction, for obvious reasons the closure blocks were kept horizontal as long as they were used for stream diversion. It was possible to form and concrete blocks Nos. 37 and 44 by taking advantage of short periods of low-water flow by ordinary construction methods. The entire discharge of the river was then diverted over blocks Nos. 38, 40, and 43. Special construction methods were needed to close, form, and concrete these blocks.

The final closure scheme adopted was to use a steel closure gate designed and purchased especially for the purpose of closing the upstream end of any one of the blocks, thus diverting the stream flow through the other blocks while a lift of concrete appromixately 5 feet high was being placed in the blocks which had been closed.

Since block No. 38 included an outlet trashrack and two outlet slide gates and outlet tunnels, and because the normal river low-water season discharge was small enough to permit diversion over only one block at a time, block No. 38 was closed and concreted ahead of blocks Nos. 40 and 43. This permitted uninterrupted construction in block No. 38.

Blocks Nos. 40 and 43 were left as the final closure blocks. Their closure was accomplished by first placing the gate across the upstream end of block No. 43, concrete in which was at elevation 825.0. Thus block No. 43 was unwatered, and the river was diverted over block No. 40. Concrete was then placed in block No. 43 to a depth of approximately 5 feet, bringing its top to approximately elevation 830, or 2 feet 6 inches higher than concrete in block No. 40. As soon as permitted by specifications, the gate was removed from block No. 43 and placed across the upstream end of block No. 40, unwatering it and diverting the entire flow of the river over block No. 43. A lift of

concrete approximately 5 feet high was then placed in block No. 40, bringing its top to approximately elevation 832.5, or 2½ feet higher than concrete in block No. 43. The operation was thus repeated, alternately diverting the river flow over the higher of the two blocks by closing the lower one and concreting it to an elevation approximately 2½ feet higher than the other.

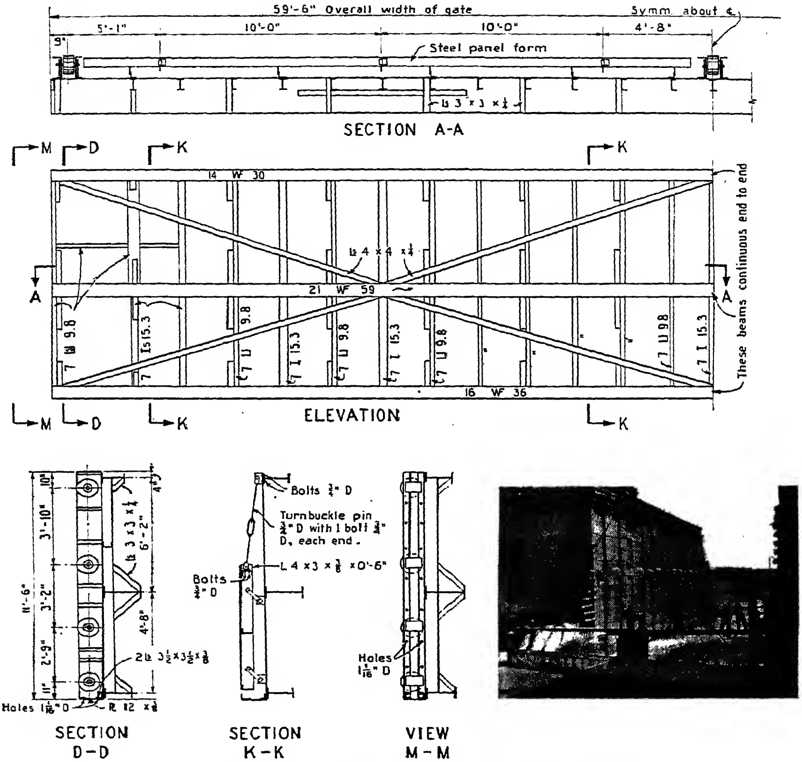

FIGURE 151.—*Closure gate and method of placing.*

Since the storage capacity of the reservoir was small at the lower elevations, the pool level rose with the closure operation until it reached elevation 860, the sill elevation of the permanent conduit outlets. After that the river flow was diverted through these conduits, and the balance of concreting in the dam was done by normal methods.

The gate used for closure purposes was 59 feet 6 inches long and 11 feet 6 inches high in over-all dimensions and carried three vertical sets of roller wheels on the downstream face, of which two were at the end and one in the middle. A rubber hose covered with a rubber belt was recessed in the downstream face along the sides and bottom

which when expanded sealed the gate against water flow. Between the rollers were two steel panels on the downstream face of the gate to serve as concrete forms for the upstream face of the dam. The weight of the steel in the gate was 32,500 pounds, and with all apparatus attached for operation its weight was 38,000 pounds. This was within the 20-ton capacity of a single cableway. Twelve-inch channels were installed along the sloping upstream face of the dam at the corners of blocks adjacent to blocks Nos. 40 and 43 to provide a track for the wheels in the gate and a smooth surface for the sealing hose. In the middle of blocks Nos. 40 and 43 to provide a smooth path for the middle wheel and also to provide a support against water pressure at this point, a 36-inch I-beam was provided with its upstream face flush with the sloping face of the dam.

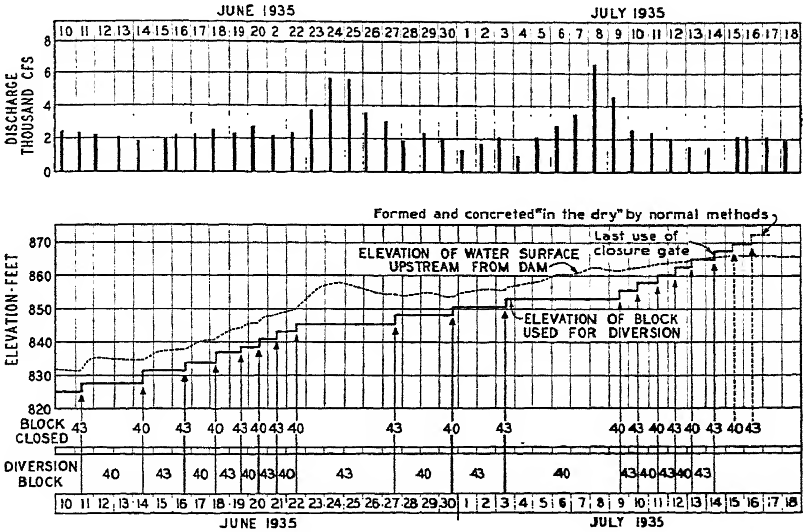

FIGURE 152.—*Closure operations.*

It had been planned initially to allow 48 hours to elapse between completing concreting in the closure block and allowing water to flow over the green concrete. This interval was cut to 8 hours when it was found that the rapidly flowing water did not damage the green concrete. Also this change was made possible when the time in which it was possible to move and set the closure gate was decreased. The cooling action of the water diverted over the newly placed concrete also made it possible to cut the time interval between successive lifts during the closure operation from the required 72 hours to 48 hours.

This method of closure was economical and involved few difficulties. The principal difficulty experienced was getting a good gate seal around the sides and bottom because of poor fabrication of the gate. Some difficulty was experienced in the upper lifts when water did not rise as high on the gate as in the lower lifts, and there-

fore did not furnish enough pressure to hold the bottom of the gate against the upstream face of the newly placed concrete. Other possible improvements indicated by experience were that the size of rolling wheels could be increased from 11 inches in diameter to 18 inches in diameter, thus decreasing the rolling resistance offered by small obstructions; that the rubber seal should be embedded about 3 inches under the steel angle at the sliding end, since some difficulty was experienced from the belt pulling out of the seat; that a seat for jacking the gate down into position should be provided in the sides of the blocks adjacent to the closure blocks; and that pumping wells should be placed well down in the gate to handle leakage past the seals.

The cost of closure equipment and closure operations is given in table 82:

TABLE 82.—*Cost of closure equipment and closure operations*

	Total cost
Closure operations—18 cycles:	
General (job pro rata)	$277.36
Setting gate	2,790.42
Sealing gate	1,422.81
Unwatering	2,169.05
Total	6,659.64
Gate, guides, and supports	
Gate, f. o. b. Coal Creek	1,936.94
Guides, supports, and installation	5,686.43
Total	7,623.37
Total cost of gate and operation	14,283.01
Recondition and prepare for storage after completion of closure	357.02
Gross cost of gate	14,640.03
Sale to Hiwassee (credit)	905.77
Total	13,734.26

Operation after closure.

After July 13, 1935, the entire discharge of the river was handled through the permanent conduits. Until March 4, 1936, there was no attempt made to store water in the reservoir as all gates were kept open except for short periods of trial operation for individual gates. The upstream water level was maintained at approximately elevation 870 except for brief intervals when the flood discharges built up a higher head on the outlet intake, after which the level again dropped. On March 4, 1936, after installation of the penstock gates was completed, the last construction operation requiring a low reservoir stage, the conduit gates were closed and the entire river flow was used in building up the reservoir storage. Thus, with the building up of reservoir storage, the river diversion and closure operations were completed.

Forecasting.

To avoid loss of time and money during construction operations resulting from such uncertainties as river conditions and the flooding and overtopping of cofferdams, a forecasting service was set up early in February 1934. During construction the river stages were forecast sufficiently far in advance to enable the construction forces

to take whatever precautions were deemed necessary to safeguard the project and the machinery in the cofferdams. After the river had been closed off, the forecasts guided the operation of the discharge gates and assisted in the determination of the releases to be made for the benefit of navigation on the Tennessee River.

Prompt forecasting necessitated the establishment of an instantaneous means of communicating rainfall and river stage reports from widely scattered measuring stations. A study of the past data indicated the rainfall and stream gages from which readings were required for forecasting purposes; arrangements were made to have the readings telephoned or telegraphed to the forecasters in accordance with specific instructions supplemented by special instructions during flood periods. This procedure was satisfactory except for isolated stations that were at a considerable distance from a telephone or a telegraph station. Accordingly, experiments were conducted to devise and test equipment that could be placed at these stations to broadcast automatically at stated intervals by means of a short-wave radio transmitter the stage of the stream or the amount of rainfall, as the case might be. One of these radio stream gage transmitters now provides reports of stages on the Clinch River above Tazewell.

When construction began there was but little information available on the subject of forecasting river stages. Work of other investigators on the principle that the stage at an upstream station has a definite relation to the subsequent stage at a station farther downstream and the unit graph method[1] were studied in detail and were eventually included in the procedure for forecasting the stages at the dam.

The earliest forecasts were made on the basis of the peak relationships. The long, narrow watershed of the Clinch River causes a flood peak to take definite shape in the upper reaches and to increase gradually in volume as the peak travels downstream. This characteristic permits the estimation of the probable downstream discharge peak by means of peak relationship charts as soon as a crest has formed at an upstream station, provided that the intensity of the storm is approximately the same over the entire watershed. The peak relation method was used to estimate the stage of the Powell River at Arthur from the stage at Jonesville, and the stage of the Clinch River at Tazewell from stages at Cleveland and Speers Ferry. The Arthur and Tazewell stages were then related to Coal Creek to determine the stage at the dam.

When the rainfall over the lower part of the basin was heavier than that over the upper part, it was necessary to predict the stages from the amount of rainfall. The rainfall method was also used when a second storm occurred while a peak from an earlier storm was approaching the dam. The best results in computing flood flows from rainfall have been achieved by the use of the unit graph method. The unit graph is based upon the theory that for a given drainage basin there is a definite total flood period corresponding to a given rainfall duration, and that all 1-day rainfalls will have the same base length of hydrograph. If a given 1-day rainfall produces

[1] Sherman, L. K., Stream Flow from Rainfall by the Unit Graph Method, Engineering News-Record, Vol. 108, pages 501–505.

1 inch of run-off over the area, the hydrograph can be considered as a unit graph for the watershed with the ordinates of run-off varying directly with the depth of run-off. Then a storm hydrograph may be constructed by using ordinates obtained by multiplying ordinates on the unit graph by the average run-off in inches and making corrections for the base flow of the river.

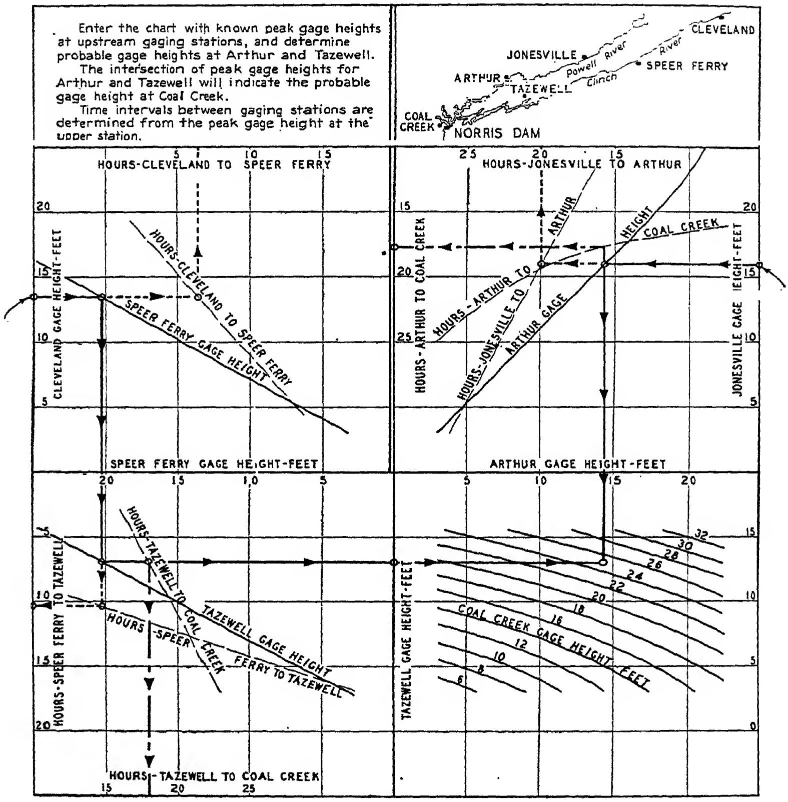

FIGURE 153.—*Flood forecasting chart.*

The unit graph method assumes that the rainfall will be approximately equal in intensity over the watershed; therefore, the area below Arthur and Tazewell and above the dam was considered as an individual drainage basin in the development of the unit graph. To obtain the best results, as close a determination of the rainfall must be made as is economically possible. As the seven existing Weather Bureau gages in and around the watershed were insufficient, gages were placed in 17 C. C. C. camps, and a number of recording gages were installed throughout the drainage basin.

From February 1934 to August 1934 the channel at the dam was only 40 percent unobstructed. Warnings were desired whenever floods exceeding 13,000 cubic feet per second were anticipated. In August 1934, when the west cofferdam was placed, warnings were desired of all floods that might overtop that structure; accordingly, forecasts were made of all discharges which would exceed 6,000 cubic feet per second.

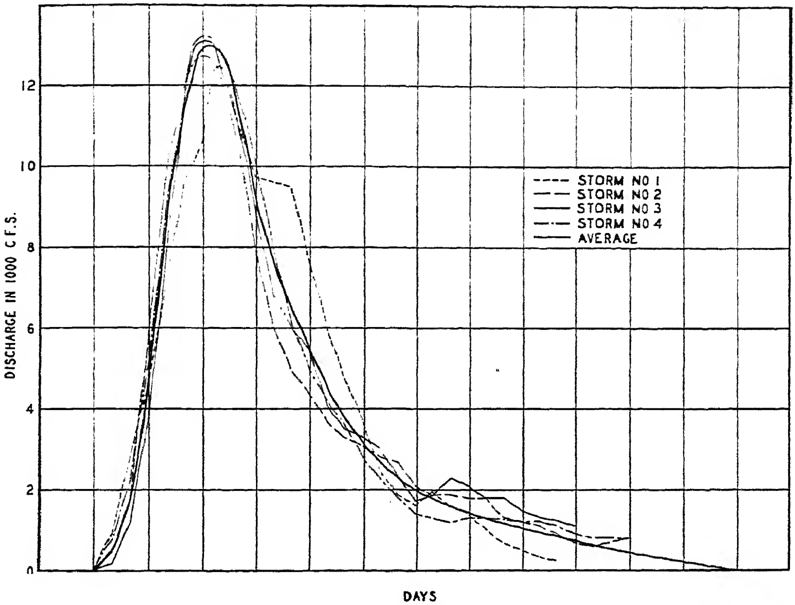

FIGURE 154.—*Twenty-four hour unit graph—Arthur, Tazewell, Coal Creek watershed.*

Any forecasting system is no more reliable than the maximum error which may occur on any one forecast. By the summer of 1935, it was possible to predict stages 36 hours in advance with a maximum error of 3 feet in stage and of 4 hours in time; stages could be predicted 20 hours in advance with a maximum error of 1.5 feet. As a storm developed, the long range forecast was always corrected with the result that the error in most of the final forecasts was but a few tenths of a foot from the actual stage.

DISPOSAL OF CONSTRUCTION PLANT AND EQUIPMENT

A decided saving was realized in the net cost of the construction plant for the dam through planning of continued equipment use on other Authority projects, combined with a study of market conditions for possible sale of released equipment. Transferred equipment was appraised on a basis of physical condition and use value. Equipment sales were consummated when a study of physical condition

and possible use showed a bid price greater than the value of the items to the Authority. Table 83 shows some of the major construction plant and equipment items transferred from Norris Dam and their approximate gross cost and salvage value.

TABLE 83.—*Gross cost and salvage value of major construction plant and equipment items*

Item	Number of each	Gross cost	Salvage value	Disposal
Shovel, Bucyrus 75-B	1	$40, 639. 12	$15, 387. 00	Chickamauga Dam.
Shovel, Marion 4101	1	49, 356. 15	18, 707. 04	Do.
Shovel, Marion 450	2	18, 348. 78	12, 541. 14	Pickwick Landing Dam.
Shovel, Lorain 75-B	1	9, 169. 91	1, 690. 12	Hiwassee Dam.
Stationary mixers	3	20, 561. 21	9, 253. 00	Do.
Dump truck, 12-cubic-yard	5	40, 254. 18	4, 758. 82	Chickamauga Dam.
Do	2	15, 425. 00	8, 013. 27	Pickwick Landing Dam.
Dump truck, 8-cubic-yard	2	10, 743. 70	2, 709. 00	Hiwassee Dam.
Stake truck, 5-ton	6	12, 767. 34	4, 770. 22	Hiwassee Dam (2). Chickamauga Dam (4).
Cement haulage units	4	22, 028. 00	6, 585. 00	Hiwassee Dam.
Tractors	4	21, 804. 08	3, 711. 00	Sold outside TVA.
Plymouth locomotive	3	11, 715. 00	5, 463. 41	Chickamauga Dam.
Do	1	3, 905. 00	2, 402. 92	Pickwick Landing Dam.
Concrete transfer car	3	15, 223. 00	4, 415. 77	Hiwassee Dam.
Concrete bucket, 6-cubic-yard	3	2, 990. 00	1, 293. 00	Do.
Hoist double drum, 100-ton	1	2, 750. 00	1, 705. 00	Do.
Mixer plant structure	1	18, 377. 00	9, 189. 00	Chickamauga Dam.
Aggregate plant (rock)	1	116, 319. 00	58, 160. 00	Do.
Aggregate plant (sand)	1	49, 797. 00	27, 800. 00	Hiwassee Dam.
Cableway No. 2	1	113, 208. 00	56, 600. 00	Do.
Cableway No. 1 head tower and machinery only	1	113, 208. 00	40, 000. 00	Sold outside TVA.
Cableway tail tower			3, 000. 00	Unassigned equipment account
Truck, 5-ton	1	2, 128. 00	857. 00	Guntersville Dam.
Light truck	2	1, 370. 00	100. 00	Sold outside TVA.
Miscellaneous items	[1] 900	333, 913. 00	157, 196. 00	Various.
Total		1, 046, 000. 00	456, 308. 00	

[1] Approximate.

The amounts given in the preceding table as original costs do not include the costs for supporting structures, foundations, labor for installing and dismantling the equipment, or any other costs that had

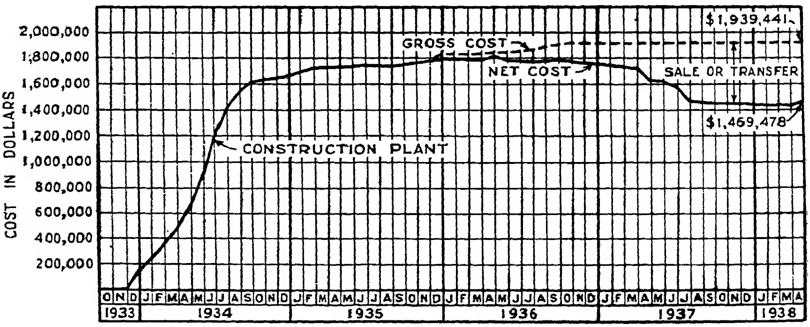

FIGURE 155.—*Plant and equipment costs—Gross and net.*

to be fully appreciated at Norris. Figure 155 gives the gross and net costs of the construction plant for the dam and includes those items which had no salvage value.

In addition to the salvage obtained from the dam construction plant, approximately 287 items of equipment were transferred or sold from the reservoir and town areas with a salvage value of $170,360. These items of equipment were used for reservoir clearance work; for highway, railroad, and other relocation work; in the construction of the construction camp and village; and for other work done in the reservoir area. The equipment used for reservoir and town work was purchased with a capital outlay of approximately $282,400.

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Chapter 6

DAM AND POWERHOUSE CONSTRUCTION

The construction activities necessary for the construction of the dam and appurtenant works are divided into three parts: first, *Organization and personnel*—how personnel was procured, personnel policies, the construction organization, and safety and training programs; second, *Materials*—dealing with the procurement, testing, and inspection of the materials and equipment necessary for the project; and third, *Construction*—including the construction plan, methods, progress, and discussions of how the difficulties encountered were solved.

ORGANIZATION AND PERSONNEL

Construction was carried out by a force account organization except in those few instances where special equipment and personnel, not normally a part of an organization of this type, were desirable or necessary for the completion of individual parts of the construction. In these cases the work was done by contract, as for example the welding and erection of the penstocks.

The Norris construction organization functioned under two main heads—the construction engineer, and construction superintendent—who in turn coordinated their work through the assistant chief engineer. As the Authority widened its scope of activity and other projects were undertaken, it became necessary to create the position of chief construction engineer, after which the construction engineer and construction superintendent were responsible immediately to him.

The principal staff members of the construction engineering organization were the assistant construction engineer, field engineer, office engineer, materials engineer, electrical engineer, foundation engineer, and cost engineer.

The construction superintendent was assisted by a staff composed of two assistant construction superintendents who alternated on day and night shifts each month, a master mechanic, chief electrician, chief rigger, general carpenter foreman, excavation foreman, foundation treatment foreman, concrete foreman, and chief clerk.

Plan of operations.

Since construction was started at a time when a concerted effort was being made throughout the country to reduce unemployment, the Authority formulated a work plan whereby a maximum number of men would be employed economically. This plan included four $5\frac{1}{2}$-hour labor shifts and three 8-hour supervisory shifts. As far as is known, this was the first time that such a program was followed, and it attracted considerable interest in the field of heavy construction. The principal shifts were scheduled as follows:

Shift	Labor	Foremen	Key foremen
1	6:30 a. m. to 12 m.	8 a. m. to 4 p. m.	6:30 a. m. to 2:30 p. m.
2	1 p. m. to 6:30 p. m.	4 p. m. to 12 p. m.	2:30 p. m. to 10:30 p. m.
3	6:30 p. m. to 12 p. m.	12 p. m. to 8 a. m.	10:30 p. m. to 6:30 a. m.
4	12 p. m. to 5:30 a. m.		

This schedule proved very satisfactory, especially because it afforded continuity of work, since at no time were laborers and supervisors changing shifts simultaneously. The general foremen worked the first or day shift and were relieved and represented by their respective shift foremen on the other two shifts.

Duing the period from the beginning of the job until August 1935 this plan was in effect generally, altho it was not practical to follow the plan completely, especially in erecting the construction plant. On September 1, 1935, the labor forces were placed on four 6-hour shifts as follows:

Shift:	
1	6: 30 a. m. to 12: 30 p. m.
2	12: 30 p. m. to 6: 30 p. m.
3	6: 30 p. m. to 12: 30 a. m.
4	12: 30 a. m. to 6: 30 a. m.

After January 1936 the general plan was altered from time to time to suit the changing conditions on the nearly completed job. Three 8-hour labor shifts were adopted in January 1936 and continued until July 1936 when only one 8-hour shift was used in completing the final details of the job.

Employment procedure.

Professional personnel.—Norris was the only TVA construction to be started immediately after the creation of the Authority. The publicity which accompanied the passage of the Tennessee Valley Authority Act resulted in applications for employment being received at the rate of well over 1,000 per day. Formal application blanks were sent to those whose letters indicated that they were qualified to render some service to the Authority, regardless of their geographical residence. When the completed forms were returned, references were checked and qualifications of the candidates were classified. As assistants were needed by the construction engineer and the construction superintendent the best qualified applicants were selected from the files thus secured. All those selected were subjected to a thorough physical examination and, for major positions, the approval of the Board of Directors.

Section 6 of the Tennessee Valley Authority Act specifies[1] that all appointments shall be made on the basis of merit and efficiency without any political test or consideration. Thus, in the selection of employees for service with the Authority, efforts were made to observe the spirit as well as the letter of this particular provision. It was recognized from the very beginning that the success of the project depended largely upon the ability to obtain the services of competent personnel at all levels.

[1] See appendix J.

Nonprofessional personnel.—When offices were opened in Knoxville in August 1933 the Authority was literally overwhelmed by thousands of applicants for labor and trade positions. Careful appraisal of the applicants interviewed in such large numbers was well-nigh impossible. To solve this problem, the United States Civil Service Commission, at the request of the Authority, administered a series of examinations in which all applicants for nonprofessional positions (laborers and tradesmen) were required to participate in order to qualify for such positions. As a means of forestalling an influx of unemployed workers from other parts of the country, residence in the Tennessee Valley and certain additional areas was required of examinees. Announcements of the examination were published in newspapers and posted in post offices in the area.

In order to insure that well-qualified persons would file applications and thus be eligible to stand the examination, personnel representatives visited approximately 175 counties in the area, which included all of the State of Tennessee and parts of Kentucky, Virginia, North Carolina, Georgia, Alabama, and Mississippi. These representatives explained the examination procedure to leaders in the counties and asked them to urge competent persons of their acquaintance to participate in the tests. Applications were made available in the post offices in the examination area. The Civil Service Commission issued admission cards for the examinations to all candidates whose blanks were filed on time and who met the residence requirements.

The examination consisted of a test of ability to follow oral and printed instructions, a simple reading test, and a mechanical aptitude test. A nonlanguage test enabled persons of limited formal education to make some score on the examination. In this manner, persons of good native intelligence but without facility in reading were not eliminated from final consideration. The fact that less than half of the persons who received application blanks actually participated in the examinations indicates that the examination requirements operated in themselves as a means of selection in eliminating many who were not particularly interested in employment with the Authority or who feared they might not qualify. When the examination results were available in percentile rank distribution, interviewers were sent into the field to talk to prospective employees. These interviewers were men familiar with the requirements of various trades, and they endeavored to judge accurately the qualifications of specific individuals. In addition, they talked with leaders and employers in various communities to check upon the experience, qualifications, and general competency of applicants, as well as to inquire into their standing in the community. The interview records were then placed in the files of the applicants, and as men were requisitioned for work, candidates were selected with due regard to special skill, experience, and character. In many instances, the candidates actually employed had acquired experience in construction work in other sections of the country.

For the most part, laborers and tradesmen were selected from the eastern portion of the examination area previously mentioned. Thus workers came from approximately 45 counties in central and eastern Tennessee, 8 counties in western Virginia, 15 counties in western North Carolina, 9 counties in northern Georgia, and 6 counties in

southeastern Kentucky. Definite quotas of employment were not assigned to these counties, but a number of employees was selected from nearly every one. Only emergency requisitions could be filled immediately as the details of this procedure ordinarily required approximately 1 week. However, this difficulty was largely overcome by the construction supervisors' anticipating their labor requirements as far in advance as possible and working in close cooperation with the personnel officers. The over-all effect of the employment procedure was that a superior staff of workers was secured. This was especially important because many foremen and other supervisors were promoted "from the ranks" and, at the completion of Norris, transferred to other Tennessee Valley Authority projects.

Salary and wage schedules.

Salaries paid to annual employees of the Authority during the first few months were somewhat in accord with rates then prevailing in private industry. However, an Executive order [2] issued by the President on November 18, 1933, established a salary schedule to be followed by the newly created emergency unemployment relief agencies. This schedule largely approximated the rates prevailing for the regular governmental departments, although there were certain differences in the higher levels. While the Authority was not bound by this Executive order [3] the Board of Directors determined to follow it for the most part. The job supervisors prepared descriptions of duties for all positions, and these were compared with positions described [4] by the Personnel Classification Board and the Civil Service Commission. In this manner, comparable salaries were paid for comparable duties and responsibilities. Wage schedules for the nonprofessional workers were established and adjusted from time to time to agree with the prevailing wages for similar occupations in the surrounding area. The hourly wage schedules are included in appendix I.

Labor relations.

During the first 2 years of construction the maintenance of proper individual and collective relations between supervisory personnel and supervised employees, especially regarding working rules, regulations, established rates of pay, and working hours, was given much time and consideration. At the instigation of either supervisors or supervised employees, complaints were investigated and differences settled. The Authority also undertook the responsibility for seeing that employees understood their rights with respect to employee organization and cooperated with employee organizations. It maintained contacts with other organized labor groups in the area and with other agencies of the Government relating to organized labor. It also undertook to make certain that labor provisions of the Authority's contracts were fulfilled.

With the formal adoption of an employee relationship policy [5] in August 1935, the approach to these problems was changed. Em-

[2] Executive Order No. 6440. (See appendix J.)
[3] Executive Order No. 6746 specifically exempted the Tennessee Valley Authority from the requirement of Executive Order No. 6440.
[4] Preliminary Class Specifications of Positions in the Field Service. Government Printing Office, 1930.
[5] See appendix I.

ployees previously came direct to the management with their grievances, and members of the staff served as intermediaries between the employees and their supervisors. The employee relationship policy, however, specifically requires that employees work through established supervisory channels, only those cases coming to the management which were appealed after failure of satisfactory settlement on the job.

Training and recreation.

The 33-hour work week, making considerable time available for employee training and recreation, led to the establishment of a program [6] of formal and informal educational and recreational activity and was well under way by April 1934. The purpose of this program was to provide:

1. Further training in the vocations in which the individual was already employed.
2. Opportunity to explore vocational possibilities and secure assistance in preparing for new vocations.
3. Job training for basic rural occupations, including in addition to those commonly associated with agriculture those occupations and trades which contribute to a more orderly and complete rural life and which might relate to a coordinated development of agriculture and industry.
4. A general educational and community program for employees and their families.

Training, wherever feasible, was associated with service enterprises developed to serve also the construction program and the town of Norris. These service projects were operated on a full-time basis by employees of the Authority, some of whom devoted their entire time to the service aspect of the work while others divided their time between training and service work. Theoretical and practical training on most of these projects was given at periods arranged so that men on all of the various work shifts could be accommodated.

Activities in this program included engineering, trades and construction training, agricultural training, recreation, home planning and management, general adult education and library service, educational and commercial motion pictures, and an elementary and secondary school.

The library, including about 4,000 volumes, provided books covering a wide range of reading interests as well as materials for use in the several educational activities. A special service was rendered the reservoir clearance crews by making books available to them as explained in chapter 7.

The community building and recreational areas, which included soft ball and baseball diamonds, and tennis, horseshoe, and volley ball courts, offered facilities for wholesome recreation activities. This program, sponsored in cooperation with an employees' recreational association, included outdoor and indoor games of all kinds, chorus, athletics, band, orchestra, dramatics, hiking, fishing, and boating.

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[6] Dawson, J. D., Progress in the Tennessee Valley, Federal Employee, 19: 7, November 1934. Reeves, F. W., TVA Training, Journal of Adult Education, 7: 48–52, January 1935.

The auditorium of the community building was equipped for showing motion pictures. Theatrical films, selected from the best available lists, covered the usual range of serious drama, comedy, musical romance, westerns, newsreels, and travelogs. The same equipment, supplemented by portable 16-millimeter machines, was used for educational films shown to increase the effectiveness of other parts of the educational program.

Safety.

A formal safety program was inaugurated in June 1934 in recognition of the need for organized control of accidents, fire, and public safety hazards on the various operations. An initial survey of safety conditions revealed interesting facts pertinent to the program. Some of these were:

1. Most of the supervisors, while outstanding men in their respective fields, had not been associated with each other prior to this project. A cross section of their individual opinions regarding the place of safety in the operations and method by which it was to be achieved revealed attitudes varying between definite opposition to an organized safety program and the continuation of the various safety policies which had been followed on previous operations.
2. Although no statistics were available for comparison, a general impression existed that the accident experience at Norris was below the average for this type of work and that the safety of operations required no further attention than was being given.
3. The supply of both skilled and unskilled local laborers experienced in heavy construction methods and practices was inadequate, necessitating considerable in-service training to develop competent men.
4. In contrast to these unfavorable factors were the extensive use of modern construction methods and equipment, excellent physical plant lay-out, high-type supervision, detailed project planning, advanced personnel policies regarding selection and placement of personnel, employee management relationship and job training, and the provision of complete medical facilities and first-aid service.
5. A study of the initial accident experience of the Authority based on available compensation records indicated that the cumulative frequency rate at Norris Dam was approximately 15 percent and the severity rate 33 percent above the national averages for similar operations, and that the frequency of accidents was increasing at an alarming rate.

When information on accident experience was brought to the attention of the Board of Directors, the Chairman and Chief Engineer issued the following statement defining the broad objectives of the Authority in the matter of accident prevention:

> We are not satisfied with the accident rate on TVA projects as shown by an analysis just completed.
>
> Accidents constitute human and economic waste. Serious accidents are more costly in the TVA than in private industry. Reduction of our rate presents an immediate problem in line directly with the social and economic purposes of TVA.
>
> The safety of TVA workers is a matter of prime importance. It is our responsibility to provide the maximum of safety in working conditions and to educate the workmen in the avoidance of accidents.
>
> * * *

We desire to set an example of proficiency in accident prevention. Your efforts toward this end will be appreciated. Should any regulations or methods suggested seem not to be practical or justified, I should like to hear from you. We are anxious to reduce our accident record and also to maintain reasonable and common sense working conditions.

To achieve these objectives, comprehensive programs in the fields of industrial and public safety were developed, embracing engineering revisions to eliminate hazardous conditions through planning and design; safety education to acquaint personnel with the causes of accidents and with practical methods by which they might be avoided; and the development of employee, management, and public cooperation in the safe performance of work and in community safety through which accident hazards were anticipated and eliminated or controlled.

Through educational means, it was sought to inculcate in persons who could best control them an appreciation of the causes of accidents and means for their elimination. Direct action by the safety inspector in the control of hazards was resorted to only in emergencies.

Specific services rendered in the field of industrial safety included:

1. The maintenance of accident records and compilation of accident statistics.
2. Consultation with design and operating units on safety standards, specifications, and methods.
3. Education of project personnel in locating and controlling accident sources.
4. Specification and approval of safety devices and equipment.
5. Project inspection leading to recommendations regarding the correction of accident hazards.
6. The detailed investigation of accidents with specific attention to causes.
7. First-aid training of project personnel.
8. Assistance in placement of personnel in occupations they could perform with safety to themselves and fellow workers.

Liberal use was made of all the facilities of the Authority in the solution of safety problems, and extensive cooperation was maintained with other Governmental agencies, technical and professional organizations, and agencies having mutual interests.

Prior to the formulation of the definite safety program, specific safety activities at the dam included the mechanical guarding of obvious hazards and the provision of safety equipment, such as safety belts, goggles, respirators, and similar devices. Specific orders had been issued covering the protection of workmen suspended over the side of forms, the safe construction of scaffolds, and the provision of mechanical protection around the primary crusher at the quarry.

Upon formulation of a definite safety program, safety posters were displayed at strategic points throughout the operations, interdepartmental safety competition was established, crew safety meetings were inaugurated, and mechanical guarding was standardized and extended to the various less obvious hazards. Discussion of safety problems was included in foremen's meetings and from these discussions detailed plans for organized safety work on the project were developed.

First-aid instruction was introduced in November 1934 by an instructor furnished by the United States Bureau of Mines. At the completion of the project, a total of 300 persons, including 51 newly

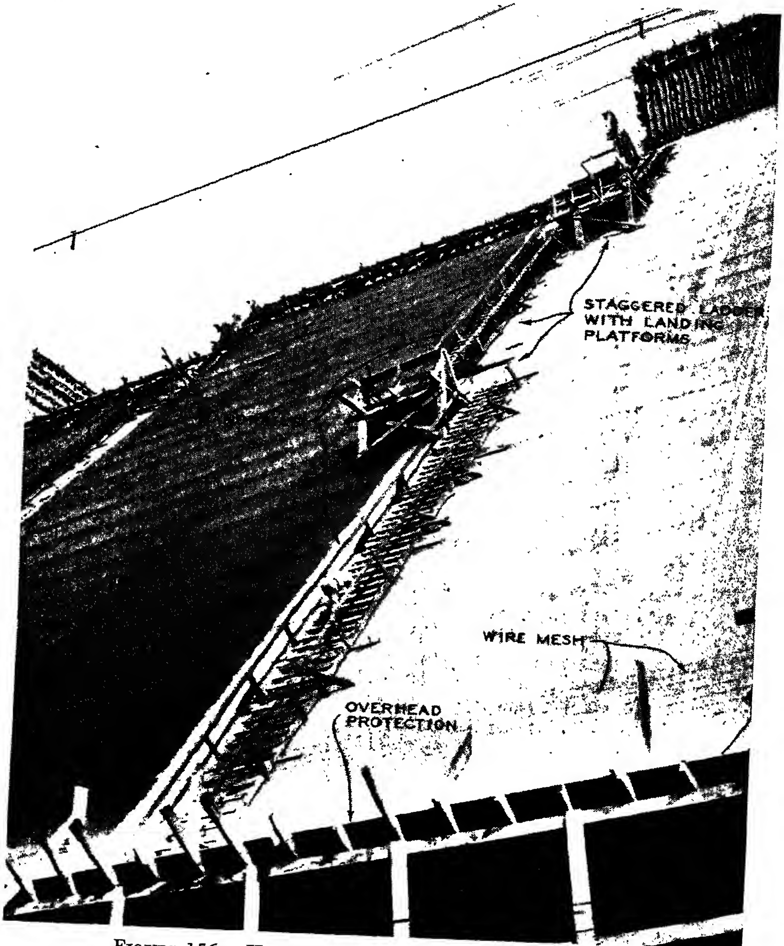

FIGURE 156.—*Handrails, ladders, and other safety features.*

qualified instructors, had taken the basic first-aid course and advanced training.

The accident experience at Norris when compared with the accident experience [7] of 38 construction firms, members of the Associated

[7] U. S. Department of Labor, Monthly Labor Review, 47: 332, August 1938.

General Contractors of America, engaged in the construction of "abutments, aqueducts, elevated highways, heavy foundations, docks and harbors, large bridges, viaducts, locks, dams, levees, sewers, tunnels, etc." (which is, in general, comparable with the work at Norris), shows the following:

	38 private contractors calendar year 1936	Norris Dam Sept. 1933–Mar. 1938
Average number employees	5,605	912
Man-hours of work	7,587,448	7,585,238
Disabling injuries	1,530	371
Fatal injuries	15	7
Permanent disabilities	130	14
Temporary disabilities	1,385	350
Frequency rate [1]	202	48.9
Severity rate [1]	46	8.78
Average days lost per temporary disability	17	26.8

[1] Weighed in accordance with standards of International Association of Industrial Accident Boards and Commissions.

The yearly records of frequency rates of accidents per million man-hours and the severity rates in days lost per 1,000 man-hours worked were as follows:

	Year ending June 30			
	1934	1935	1936	1937
Frequency rate	64.6	62.5	32.6	7.1
Severity rate	8.55	14.44	3.53	0.03

Norris Dam was designated "Honor Roll Company" by the National Safety Council for maintaining the lowest frequency rating during 1936 of the five organizations of comparable size engaged in dam construction operations.

The above table shows a steadily decreasing accident rate as construction progressed. Although this improved accident experience represented a savings to the Federal Government of many thousands of dollars in direct compensation and medical expenses, the greater monetary value of this experience was reflected in increased job efficiencies made possible through uninterrupted work schedules, low labor turnover, less damaged equipment, and less spoilage of materials. In addition to these direct savings, the conservation of human resources represented by fewer accidents constitutes a definite contribution to the achievement of the broad objectives outlined in the Tennessee Valley Authority Act regarding the social and economic development of the region.

Medical service activities.

During the construction period, medical service was rendered through three field organizations or medical units. These units were located and designed to serve the dam, the town, and the reservoir area. These field units were directed from the office in Knoxville, Tenn.

The medical service program rendered the following major services:

1. Physical examinations and classifications of employees and applicants selected for employment.
2. Immunization of employees against typhoid fever and smallpox.
3. Care, treatment, and handling of employees injured while in the performance of duty.
4. Emergency care of non-service-connected minor illness occurring to employees while on duty.
5. Prophylaxis and treatment of gonorrhea and syphilis.
6. Medical care of employees and their families residing in areas remote from medical facilities.
7. Preparation and handling of compensation claim forms.
8. Emergency care of injuries and illness occurring to non-employees visiting construction projects.

Medical service was provided in the area through field units from November 10, 1933, to December 1, 1936. During these 3 years 98,679 services were performed. A total of 37,774 services, or 38 percent, was performed at the dam in support of the dam construction activities; 40,911, or 42 percent, were performed at the town, primarily composed of hospital care and medical activities in support of the town construction activities; and 19,994, or 20 percent, were performed in the reservoir, primarily in support of reservoir preparation activities.

Of the total area services, 40 percent were performed for employment physical examinations and immunization; 15 percent for service-related conditions, such as medical care of employees and their families and treatment of venereal diseases; and 45 percent were performed in the actual treatment and handling of service-connected injuries.

During the construction period there was an average of 2,736 employees per month and an average of 2,741 medical services per month, indicating an average of one service per employee per month.

The total cost of these services for the construction period was $125,181.24, or an average cost of $1.27 per man per month.

The cost analysis given above covers only actual expenditures by the Authority and does not include expenditures of other Federal funds by the United States Employees Compensation Commission for compensation and medical service beyond that provided by the Authority.

MATERIALS

The procurement program is one of the vital factors in the efficiency and economy of a construction project. Its function is to purchase and deliver to the job at the proper time the required materials and equipment at a minimum cost. The schedule of construction and the quality and cost of the final structure are very closely related to and dependent upon the activities of the procurement forces. The procurement problem in connection with this project consisted not only of the acquisition of materials and equipment, but also the development of a permanent purchasing system for the Authority as a whole.

Development of purchasing system.

The original Tennessee Valley Authority Act did not contain any provisions relating to purchases of materials and equipment. The Board of Directors took the position that in general the Federal Purchasing Statute, section 3709 of the Revised Statutes, was not applicable to the Authority. They decided as a matter of policy, however, to comply with its provisions when feasible. Studies were made of the policies and procedures in procurement used by various governmental agencies and by private corporations whose purchasing activities merited attention. A purchasing system was outlined for the Authority which incorporated the most adaptable features of the systems studied. The adopted plan placed the procurement for all projects under the control of a central purchasing organization.

Purchases were made on the basis of least final cost to the project and the Authority. The practice of award to the lowest bidder meeting all conditions of the bid and complying with the specifications was adhered to where feasible, but quality and dependability were also determining factors. This procedure led to the rejection of low bids in a few cases where initial price was not the prime requisite. The awards for the primary crusher and the hammer mills were examples of such action. It was essential that proved equipment be placed at these strategic points, as a breakdown in either of these items would have shut down the entire construction plant, and the resulting loss would have exceeded greatly any difference in prices. This practice, while sound from an engineering point of view, did not conform to Federal procurement regulations. Considerable time and money were expended in defending this policy. The Comptroller General finally upheld the Authority in the cases where exception was taken by dissatisfied bidders, but insisted that legislation be enacted to regulate the Authority's purchases.

Purchasing system under amended act.—The controversy between the Comptroller General and the Authority was brought before Congress during a hearing on the amendment to the TVA Act. The final outcome was that a statement of TVA purchasing policy was incorporated in the amended act.[8]

The amended act gave the Authority the discretionary powers necessary to direct its purchasing for the greatest benefit to the project and allowed for the full exercise of procurement and engineering judgment in purchasing the Authority's requirements on a business-like basis. Under the amended act it was possible to acquire sound and dependable equipment and to secure for the permanent structures materials, machinery, and workmanship which would result in the greatest service per dollar expended.

The general authority to purchase and contract for materials, equipment, and supplies was vested in the Board of Directors. This authority was delegated by the Board to the director of purchases and his authorized assistants. A procurement organization, responsible to the director of purchases, was established in order to:

1. Coordinate the using departments and the various warehouses, converting requisitions into transfers from stores where possible.

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[8] See appendix J, sec. 9 (b).

2. Approve, or have approved by the director of purchases, all purchase requisitions.
3. Prepare specifications or approve them from the standpoint of nomenclature, form, exactness, and degree of limitation.
4. Initiate and advertise invitations to bid.
5. Prepare and distribute invitations to bid.
6. Open and record bids.
7. Make awards.
8. Prepare and sign purchase orders.
9. Negotiate and sign contracts.
10. Perform or coordinate the enforcement of inspection and testing requirements.
11. Route, coordinate, and expedite the shipping of purchases.
12. Maintain the Authority's mailing list of bidders.
13. Conduct standardization studies of material, equipment, and supplies.

Purchasing procedure.—The three general methods of purchasing materials, equipment, and supplies were to purchase after advertising, to purchase in the open market, and to purchase small items without competition. The methods used depended upon the circumstances surrounding the individual purchases. The procedure for each method was based on purchase regulations approved by the Board of Directors and on the coordinate requirements of the organization concerned with the procurement.

Advertising was employed for original contracts involving more than $500 which did not entail an emergency, the purchase of repair parts, or the securing of supplemental equipment, materials, or services. Requisitions were initiated at the using office, forwarded to the budget officer for authorization and to the director of purchases for approval. Specifications were prepared or approved and invitations to bid were drawn up, advertised, and distributed to prospective bidders. Advertising was conducted through newspapers, trade periodicals, circulars or letters sent to vendors, and by notices posted in public places. The minimum time of advertising varied from 1 to 3 weeks, depending upon the nature of the purchase, and was extended when necessary. Formal bids received after advertising were opened, read, and recorded in public. Informal bids were opened at the appointed time and recorded, but not read in public. They were evaluated on the basis of delivered cost. Analysis of the involved bids requiring engineering judgment and recommendations of award were submitted by the engineer in charge. Awards were approved by the director of purchases and notices of award were sent to successful bidders. The final contracts were prepared and distributed to the successful bidders for execution and return. Shipping instructions were prepared and forwarded to the contractor. The necessary inspection and testing were carried on, and final acceptance was made by the office initiating the requisition. Final payments were made by the Authority according to the terms of the individual contract.

Purchases involving less than $500, purchases of an emergency nature, and purchases having a single source of supply were made in the open market when necessary in much the same manner employed by private business concerns.

Small items involving less than $25 were purchased from wholesalers and jobbers without competition. A staff of purchasing agents was maintained to handle this work along with other purchasing operations.

Each of these methods has its own particular advantages and disadvantages. Purchase after advertising gave access to the widest market and made possible the lowest prices. This method, however, required a considerable time. Purchase in the open market had a distinct advantage of time element but did not give access to as wide a market. The purchase of small articles without competition eliminated much of the procurement cost on items where printing costs alone would have been a large proportion of the total cost. The three methods, used discriminately, afforded excellent means of satisfying the purchasing requirements.

Summary of procurement activities.—The procurement program included the purchase of approximately $13,700,000 worth of materials, supplies, and services for the construction plant, dam, powerhouse, switchyard, transmission lines, and activities in the reservoir area. Purchases of land were made separately and are described in chapter 7. Concrete aggregates were manufactured at the site, and the access roads and several other jobs were contracted for in the finished state. The great bulk of the materials, however, were purchased directly.

In addition to actual purchasing and contracting, every practical means of cost reduction was investigated and utilized. Probably the greatest tangible savings were effected through the full utilization of land grant freight rate reductions. This item alone amounted to a saving of approximately a quarter of a million dollars. Where possible all equipment was standardized and the Authority's requirements adapted to regular market items. This resulted in quicker delivery and better facilities for exchange of equipment as well as lower costs. Studies were made of cement production costs, and a reduction to Norris of approximately a quarter of a million dollars was effected through negotiations [9] with cement companies on the basis of requirements for all the Authority's projects over a long-term period. Specification studies were made to eliminate all unnecessary costs for additional requirements. Procurement activities were coordinated, the organization centralized, and constant studies carried on in the interests of better efficiency, economy, and service.

An exceptional feature of the Authority's purchasing operations was the participation in existing governmental contracts made by the Procurement Division of the United States Treasury Department. This practice made it possible on several items to obtain prices not otherwise available and to eliminate the duplication of the efforts of another agency. The purchase of lubricating oils, effected through participation in the Navy Department contract, is an example of such action.

Market conditions during construction.—Procurement of major items started in the winter of 1933-34 when construction costs were just emerging from a 15-year low, reached in 1932. The index average throughout the procurement period was approximately 190 as compared to 158.4 at the beginning of 1933. These figures, while not entirely applicable to the project, reflect general market conditions and show the consistent upward trend during the construction period.

[9] See appendix E.

A large part of the purchases was made in a rising market, and manufacturers naturally included a margin of protection in their bids. Contracts were designed[10] to alleviate this condition where possible but could not entirely eliminate it. Detailed analyses of bids were employed to uncover such items, and rejection of bids followed. Identical bids were received in a number of cases, especially during the existence of the National Recovery Administration. If they were acceptable, the successful bidder was determined by drawing. No

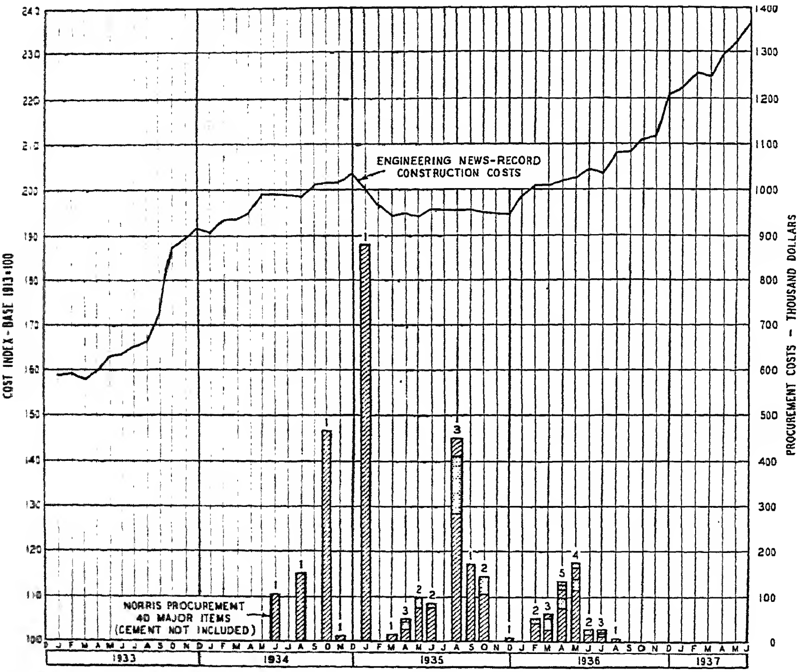


Figure 157.—*Engineering News-Record construction cost index, 1933–37, and its relation to procurement. (Number above each bar indicates the number of major items purchased that month.)*

effort was made to capitalize unfairly on market conditions at the expense of vendors; rather, efforts were directed toward minimum prices consistent with conditions at the time of purchases.

Inspection and testing of materials and equipment.

A central organization was established in November 1933 to conduct the testing and inspection of large quantities of material and machinery required for the construction of the project. By setting up a materials testing and inspection organization virtually independent of both the design and construction divisions, it was con-

[10] For example see appendix G.

sidered that such service would be more impartially and uniformly rendered. To all intents and purposes, the organization operated essentially as other laboratories and testing services in private practice and made all necessary inspections and tests, after which a certified report of all facts ascertained was rendered.

Organization.—Two separate staffs were organized to carry out this testing and inspection. The first was charged with shop inspection, including the testing of material entering into the finished product, as well as the fabrication and manufacture of parts. Regular reports were submitted on the progress of shop work, and assistance was given in expediting shipments. District offices were set up in strategic manufacturing centers in the United States from which inspectors were detailed to cover the various factories in which equipment and materials were being manufactured.

A second staff was charged with the inspection and testing of all cement for the dam and for the various work carried on under contract in the reservoir. This organization also did the necessary testing of any concrete aggregate purchased under contract. Concrete aggregates for the dam, however, were produced by the Authority at the site, and all tests for such aggregates and the concrete in the dam were made by the construction organization.

Functions.—In inspection it was not only necessary to determine the physical and chemical qualities and the strength and suitability of materials required for construction, but also to certify fulfillment of the requirements of the specifications under which purchases were made. Where a contract specified that certain delivery dates were to be maintained it was frequently necessary to expedite shop work to the extent of calling the contractor's attention to delivery dates and making regular progress reports to both the construction and design forces. Under the method of procurement of materials by the system of public advertisement and competitive bidding, it was necessary that advance specifications be prepared and attached to all invitations for proposals and that all bidders be required to submit quotations upon identical materials and equipment. This policy, therefore, necessitated close inspection of all materials furnished and work done under contract in order that a final certificate of inspection and approval, stating that the materials and equipment delivered fully met the requirements of the specifications under which they were purchased, could be issued. No approvals for payment were made without such certification.

Specifications themselves were prepared by the division originating the purchase request. All specifications, however, were carefully checked so that conflict and misinterpretation in the requirements might be avoided and also that the method of preparation and manufacture specified or implied would conform to standard commercial practice. Work of the inspection and testing organizations was further extended to include necessary consultation service in the preparation of specifications for both material and equipment and to advise upon the proper selection of materials for the purpose intended and customary commercial methods of preparation and manufacture so that such purchases could be made readily and economically.

Procedure for testing and inspection.—In general, after a review of the field requisitions it was decided whether inspection would be

made in the contractor's shop or after arrival at the project. A statement was then incorporated in the invitation to bid stating how inspection would be made.

Ordinarily from 4 to 10 weeks elapsed between the issuance of the invitation and the final award of the order or contract. Such awards were usually made by telegram and confirmed by letter, and the formal contract signed later. Immediately following the telegraphic award, instructions were mailed to the contractor stating that the materials or equipment would be subject to shop inspection and that the quality and fabrication of all materials must be in accordance with the governing specifications.

Instructions were also sent to the contractor to advise the Authority when an inspector would be needed at the plant. He was also required to furnish copies of all orders for materials so that, if required, inspection could be made at the plants of all subcontractors. Where materials were not standard or were of such unusual nature as not to be readily obtainable, salient features of the specifications were emphasized in order that no misunderstanding of the requirements might arise later.

The inspector assigned to cover the work was furnished a complete memorandum of instruction, including copies of all contract documents, prints of necessary drawings, and such other information as might facilitate proper inspection of the work. Where shop drawings required the approval of the sponsor engineer, signed copies of such drawings were furnished the inspector as soon as possible.

A rigid shop inspection was made of all permanent equipment. The specifications governing such equipment set forth the requirements in detail, and the contractor was required to furnish copies of all approved shop drawings and copies of all purchase orders placed for materials. The more important items were inspected at the place of manufacture so that only materials suitable for fabrication need be delivered to the contracting plant. As soon as fabrication started, a resident inspector arrived at the plant and followed the job to completion. Upon completion of shop inspection, in whole or in part, the resident inspector released the order for shipment. Where detailed inspection of both mechanical and electrical equipment was necessary, the mechanical resident inspector first supervised all mechanical tests and was then assisted by an electrical inspector for testing and approving the electrical details.

Shop inspectors' reports covering general contracts were submitted once a week or oftener as required. Initial reports dealt with the preliminary stages of manufacture and covered such items as approval of drawings, percentage of materials ordered, percentage of materials received and on hand, and other similar items. As fabrication proceeded, the shop reports set forth, as briefly as possible, all pertinent facts so that all interested persons were constantly informed of the status of shop work under any particular contracts or purchase orders. Close cooperation was maintained at all times with other branches of the work.

When advanced shipment of any part was desired, the resident inspector ascertained if it would be possible for the contractor to comply with the request without incurring additional expense. Where instructions regarding routing and bills of lading were already fur-

nished, the Authority's traffic supervisor was contacted so as to make the necessary adjustments in shipping schedules. Every effort was made to coordinate fabrication, inspection, and delivery.

In inspecting construction plant equipment, an experienced machine inspector was sent to the factory or warehouse to examine carefully the equipment and check drawings and specifications furnished. A thorough mechanical inspection covering both the workmanship and general condition was made. Preliminary appraisals and inspections of used equipment were sometimes made prior to purchase. Where reconditioning was necessary, a second inspection was made after all repairs and replacements and before approval for shipment.

At the completion of the contract a formal certificate of inspection and approval of items furnished under the contract was issued. Final payments could not be made without such certificates of approval. Triplicate copies were issued and attached to all formal contract documents as a matter of record.

Cement tests.—Since a modified portland cement embodying both the quick hardening qualities of ordinary portland cement and relatively low heat generation was specified for use in mass concrete for the dam, special tests for this type of cement, including chemical tests and the usual standard physical tests, were required. A complete physical and chemical testing laboratory was set up in space leased from the University of Tennessee and cement from several mills was tested. The full capacity of the cement testing laboratory probably exceeded that of any commercial laboratories then in operation in the United States.

Routine chemical and strength tests were made for each 800 barrels of cement. Physical tests for soundness, specific surface, and initial and final set were run at first for each 200 barrels and later for each 400 barrels purchased. Three- and seven-day compressive strengths were run for each 800 barrels, and, in addition, 28-day compression strength tests were made periodically.

Insofar as possible, the standard methods of tests of the American Society for Testing Materials were followed. Specific surface, or fineness, was determined by the Wagner turbidimeter. A special cork-insulated, air-conditioned curing room for storing specimens was built in which an even temperature of 70° F. and a constant relative humidity of 95 to 100 percent was maintained.

Close plant control was required for the manufacture of type B cement as specified by the Authority. During the early stages of production, considerable technical assistance and cooperation was given to the various manufacturers in order that the final product would fulfill the requirements. Inspectors were stationed at the mills and furnished with detailed instructions for sampling, shipping, and reporting. Cement was sampled by either the ball method, the mill-stream grab method, or the mill-stream composite method—depending upon the facilities at the plant. In each case samples for each 200 barrels manufactured were taken under the first contract and for each 400 barrels under later contracts. The samples were sealed in airtight containers and forwarded to the central laboratory for testing. Bins were kept sealed until the time of shipment or rejection. Plant inspectors broke the seals, inspected outlets, and supervised the loading and shipment of all cement. All cars were required to be

properly identified with inspection cards, and reports of all shipments were issued. All cars, bins, outlets, inlets, conveyers, and other equipment were required to be periodically inspected so as to prevent any possible contamination of the cement by foreign material.

Periodical chemical analyses were made of all samples to determine quantitatively the following:

Loss on ignition.	Ferris oxide.
Insoluble residue.	Alumina.
Sulfuric anhydride.	Lime.
Silica.	Magnesia.

The amounts of tricalcium aluminate, tricalcium silicate, dicalcium silicate, tetracalcium alumino-ferrite, and calcium sulphate were computed. Occasional determinations were also made of heat of hydration and specific gravity.

At times it was necessary to make additional cement tests and investigations, such as the rate of heat of hydration, Merriman soundness determination, Ira Pal's water test, and the blotter test for soundness. Comparative tests were also made by interested manufacturers and by outstanding laboratories, so as to check both laboratory methods and manufacturing processes and promote uniformity of product from the several mills.

Tests on paints and bituminous materials.—Chemical analyses of various paints were also made, among which were red lead, blue lead, white lead, aluminum, and Glyptal synthetic resin paints and varnishes. All paints were usually analyzed and checked against the specifications for pigment and vehicle composition and also to determine probable solution resistances, drying time, stability of brushing, and spreading. Bituminous materials such as emulsified asphalt, coal tar, creosote oil, and premoulded expansion joints were also analyzed. Such materials were checked for such specific items as distillation, specific gravity, penetration, asphalt content, loss on heating, ductility, solubility, absorption, distortion, and brittleness. An investigation of tar enamel primers for use as under-water paint on the drum gates and other steel work was also conducted by the laboratory.

Aggregate tests.—Concrete aggregates manufactured at the dam were periodically analyzed for the same constituents as determined in the chemical analysis for portland cement, tests for such other determinations as those for carbon dioxide, moisture, and other less important substances.

Tests[11] on concrete aggregates used for concrete in the dam and on the quality of concrete produced were made in a complete concrete testing laboratory located in the field at the dam site. For various minor projects throughout the reservoir, however, including the relocation of highways, railroads, and other utilities, regular tests were conducted on samples of concrete and aggregate sent to the central materials testing and inspection laboratory. Printed instructions for making the routine field tests and securing samples were prepared and issued to all field inspectors.

Chemical analysis of metals.—Chemical analyses were made of practically all structural steel required for cranes, generators, turbines,

[11] See appendix E.

gates, concrete reinforcement bars, and other metal work. Quantitative analyses were made of such elements as carbon, phosphorus, sulfur, and also such infrequently occurring elements as chromium, nickel, manganese, silicon, molybdenum, and copper. Practically all nonferrous alloys such as zinc spelter, bronze, brass, babbitt metal, copper, aluminum, and lead were analyzed for content of copper, tin, zinc, aluminum, antimony, iron, phosphorus, silicon, lead, and other elements.

Miscellaneous chemical tests.—Miscellaneous chemical tests were made of a number of small items such as:

Tarpaulins.
Wiping cloths.
Cotton waste.
Salt tablets for drinking water.
Metal boundary signs and bench marks.
Rubber seals.
Waterproofing compounds.
Duck.
Woven wire and chain link fencing.
Poultry netting.
Staples.
Electrical conduits.
Cables.
Calcium chloride.
Corrugated culvert pipe.
Galvanizing on barbed wire.

A chemical examination was made of the air sampled in a tunnel to determine the amount of carbon monoxide present from construction operations. The carbon dioxide content of the gas generated by the fire extinguishing system for the protection of the generators was determined. An investigation was made of the spontaneous combustion of sulphuric acid and sodium cyanide used in fumigating buildings after this condition caused the destruction by fire of one of the dormitories at the dam.

CONSTRUCTION OPERATIONS

The construction program was carried out generally in three main steps: First was the preliminary construction operations period, which included clearing the site, constructing plant roads and buildings, stripping the quarry site, building cofferdams, and constructing the main plant features. Second was the principal construction operations period, during which time the plant was utilized in producing the finished structure. Third was the final clean-up and dismantling period, during which time the main plant features were dismantled and the finishing touches added to complete the project. Although these steps overlapped during the actual operations, the construction scheme was planned largely on this basis.

In general, construction operations followed the scheme of river diversion with excavation being started as soon as a cofferdam had been unwatered. Shallow grouting was done concurrently with the excavation as the various areas were opened up, and concrete was placed upon the foundation as soon as possible. Curtain grouting and reservoir rim treatment were carried on throughout the entire period.

Such items as the powerhouse substructure, penstocks, discharge conduits, spillway apron, and training walls were built concurrently with the gravity section of the dam, while the powerhouse superstructure, installation of permanent equipment, switchyard, spillway

gates, and bridge were among the last operations completed. The earth fill and core wall to seal the east abutment were also completed toward the end of the job.

FIELD ENGINEERING

The work of the field engineering organization included a number of duties of vital importance to the successful completion of the project. Most important was the establishment and maintenance of a system of control lines, the lay-out of the construction plant, and the lay-out of the permanent structures. Other duties such as inspection of reinforcement, measurement of quantities, and recording and filing of field data and job progress, were also performed.

No attempt is made to describe all the field engineering work since much of it was of a routine nature differing but little from that experienced on other large dam projects, but the details of the more unusual problems encountered and the methods by which they were solved are given.

Organization.

During the preliminary construction activities the personnel of the field engineering organization consisted of a maximum of 16 men, including 3 four-man parties and 2 shift engineers. As operations reached their peak, this organization was rearranged to form the following:

First shift, 7 a. m. to 3 p. m.	One 3-man checking party. Three 3-man lay-out parties.
Second shift, 3 p. m. to 11 p. m.	One 2-man lay-out party.
Third shift, 11 p. m. to 7 a. m.	One assistant field engineer.

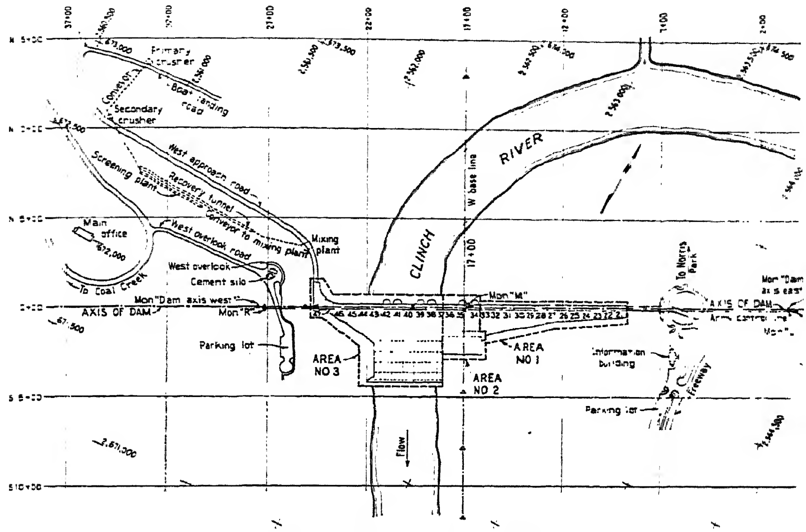
For purposes of dividing the work of the lay-out parties, the dam was divided into three areas, one of which was assigned to each of the lay-out parties on the first shift. Area No. 1 included the east abutment; area No. 2 the powerhouse, headworks, and blocks 34, 35, and 36; area No. 3 the spillway section and west abutment. The area engineers were in direct charge of all field engineering within their respective areas. The checking party that worked on the first shift provided an independent check on all field work performed by the lay-out parties. Close cooperation between the lay-out parties, construction forces, and the checking party greatly reduced errors and saved considerable time and expense.

All lay-out work not completed by the first shift was finished by the second shift party. This work was kept at a minimum since darkness increased the difficulty of lay-out and checking work. During the third shift one man was available if needed. He devoted most of his time to routine duties such as recording the starting and completion of concrete pours, entering concrete yardages, reading river or rain gages, and making various special computations.

Control lines.

Considerable work had been done at the site by the United States Army Engineers prior to the passage of the Tennessee Valley Authority Act. This work included topographic surveys, core drill explorations, and establishment of base lines and bench marks. A ma-

jor control line had been established by the Army in a general east and west direction across the river and three monuments located: Monument L on the east hill, Monument M near the east bank of the Clinch River, and Monument R on the west hill. This line had a geodetic bearing of N. 64°05′50″ E. but for simplification was called an east-west line for the dam rectangular coordinate system.



TIE BETWEEN NORRIS DAM CONSTRUCTION; RESERVOIR SURVEY; U.S.E.D. AND U.S.C.&G.S.								
MONUMENT	NORRIS DAM CONSTRUCTION		RESERVOIR SURVEY			U.S.E.D.		U.S.C.&G.S. 1936
	CO-ORDINATES	ELEVATION	CO-ORDINATES TENN. LAMBERT PROJ.	ELEVATION	GEOGRAPHIC POSITION	CO-ORDINATES	ELEVATION	ELEVATION
"R" Destroyed	0+04.282-S 26+93.22-W	1199.476	N= 672,050.58 E=2,561,902.93	U.S.E.D. Elevations accepted	Lat 36°13′23.63″ Long 84°05′41.85″	993.23 R	1199.386	——
"M" Destroyed	0+04.282-S 17+00.00-W	843.655	N= 672,501.8 E 2,562,787.8		Lat 36°13′27.92″ Long 84°05′30.95″	0+00	843.655	——
"L"	0+04.282-S 0+53.566-W	1153.401	N= 673,249.76 E=2,564,254.47		Lat 36°13′35.03″ Long 84°05′12.87″	1646.53 L	1153.361	——
U.S.C.&G.S. BM Norris Dam	——	1061.75	——	——	——	——	——	1061.865
Axis of Dam	0+00 [illegible] & S [illegible]	——	Geographic Bearing N [illegible] Grid N [illegible]			——	——	——

FIGURE 158.—*Field control system.*

All lay-outs for the dam and powerhouse were based on two major control lines. One east and west line was established in the field from information furnished by the Bureau of Reclamation, 4.282 feet north of the Army Engineers' control line. This was used as station 0+00 for all north and south chainage.

The north and south control line at right angles to the axis was established through Monument M and was called 17+00 west in order that 0+00 would fall outside of any point in the construction area. Additional semipermanent monuments and temporary targets were placed throughout the area to facilitate layout work during the construction period. The axis of the dam was extended west and a grid system laid out for the entire quarry and construction plant area.

The reservoir was mapped and control points computed on a grid of plane coordinates. Station 0+00 N. and 17+00 W. on the dam coordinate system was the same as N. 672,505.6, E. 2,562,785.8 Tennessee Lambuth on the reservoir survey. This point has a geodetic position of latitude 36°13′26.953″ and longitude 34°05′30.976″. The grid bearing of the dam axis on this grid is N. 62°58′50″ E.

Bench marks.

Bench marks were established by the United States Army Engineers from the United States Coast and Geodetic Survey, 1912 adjustment. Monument M with an established elevation of 843.655 was used in establishing vertical control for the dam. Precise levels were run to the two monuments established on the dam axis. From these points differential levels were run to Monuments R and L and other bench marks were established for the construction plant and quarry areas.

It was necessary to make frequent checks between each of the three lay-out parties to insure proper tie-in between the areas. The checking party also carried its own individual levels and furnished a valuable check on all levels, checking each party's levels as well as checking between parties. Periodic checks were made between bench marks of the four parties and any discrepancy was immediately adjusted. None of the checks ever showed more than a few hundredths of a foot difference and usually checked within 0.001 foot. Particular care was taken in setting elevations for installation of mechanical equipment to insure a good working tie-in.

In 1936 the United States Coast and Geodetic Survey ran a line of precise levels across the dam, establishing a permanent bench mark on the north sidewalk of block 21 near the west end of the observation platform and at the east end of the dam. The elevation established was 1,061.865 as against 1,061.75 from Norris Dam levels, a difference of 0.115 foot. All elevations appearing on the record drawings refer to the levels established by the Authority during construction.

Chain testing monuments.

Two heavy concrete monuments, 100 feet apart with the tops at the same elevation, were poured in place at a convenient location near the field office. These monuments had a 4-foot square footing and a 12-inch square column projecting about 18 inches above the ground, with a brass plate set in the top of each. A wooden frame was constructed between the two monuments with a continuous surface about 4 inches below the brass plates. By using a 2- by 4-inch board on edge, a chain could be supported throughout, and by using blocks it could be supported at any desired interval. Light springs with a hook and thumb screw to hold and adjust the chain and spring tension balances were placed back of each monument. A standardized 100-foot steel tape was used to establish a standard 100-foot distance on the monument. Each of the 100-foot tapes used on the job was carefully compared with the standard distance under as widely varying temperatures as possible and the temperature corrections for each tape established.

Field engineering activities.

Most of the work done was not of an unusual or difficult nature, although it required careful and accurate work by the lay-out and checking parties. Such items as construction plant lay-out, cofferdams, quarry and dam site cross sections, river, rainfall, and spring gages, field records, forms, and progress charts, deviated very little from normal practice on this type of work. The work relating to some of the permanent features is of greater interest and this will be covered in detail.

Power intake trashracks.—The trashracks for both the discharge conduits and the penstocks are semicircular in plan and are located on the upstream face of the dam. The radial center, 25 feet 6¼ inches north of the axis and on the center line of each penstock, was set at elevation 850, and control points were readily accessible in blocks 34 and 35. This point was plumbed up with a heavy bob as the work progressed. Using the center line of a target established on the opposite hill as a foresight, the column angles were turned off and offset points set for column construction and reset for setting anchor bolts for trashrack guides.

As the heights increased, the radial points could not be plumbed up fast enough or accurately enough by suspending a bob over the point. Therefore a 25-pound bob, suspended on a piano wire from a small reel, was set in oil on an offset line a few inches north of the point. This maintained an accurate line north and south and by using the center line target and lead plugs in the upstream face of the dam, the radial points could be quickly and accurately set for each lift. This point was moved to the top of the portal beam at elevation 890, as the work progressed. Elevations were maintained by carrying bench marks up the face of the dam with a steel tape. These elevations could be checked at any time from the original bench mark established at the start. The trashrack guides were steel beams spanning two openings; therefore, the anchor bolts had to be set accurately both for line and grade. No difficulty was experienced in setting any of the 64 sections of guides.

Penstocks.—Each 20-foot section of penstock pipe was accurately located for line and grade and then checked for roundness. A micrometer extension gage was made in the field for this purpose.

Powerhouse.—There were two major control lines in the powerhouse area, the 17+00 W. line running north and south; and the center line of units, 191 feet 6 inches south of the dam axis, running east and west. These two lines were maintained during preliminary construction by concrete monuments and lead plugs, set as the work progressed in the concrete and foundation rock. It was impossible to maintain any of the points as usable reference points throughout the job, but new points were always set before the old ones were destroyed or became inaccessible. These points were usually set by the lay-out parties and checked by the checking party to eliminate mistakes and avoid unnecessary confusion.

Additional north and south reference lines were established on 20-foot centers from column 1 to column 11, column 6 being 1 foot 6 inches west of the 17+00 W. base line. Likewise the center lines of units Nos. 1 and 2 were established by reference points. These lines

were maintained by lead plugs and painted targets on the electrical bay support steps on the downstream face of the dam and by lead plugs set in the rock of the tailrace. As concreting progressed, these points were transferred and kept continually in an accessible location. East and west control lines were established by chaining from the center line of units and maintained in much the same manner. These lines were designated as A, south wall of the powerhouse; B, north wall; and C, north wall of the electrical bay. This system provided a grid with ample control points coinciding with reference lines shown on all plans. Care was taken to see that control lines to be used later in setting equipment were established inside of the powerhouse before walls were constructed. After the structural steel was placed and the first section of wall poured, some of the control points were transferred to the steel columns, using a center punch mark for the point.

When the powerhouse excavation had been completed, a permanent bench mark was established at the east side of the tailrace about 50 feet south of the powerhouse. It consisted of a 1- by 1- by 18-inch steel bar grouted into rock in a niche along the east vertical face of the tailrace about 3 feet above the floor. This was called bench mark A and the elevation was established as 806.652. It was used as the governing bench mark for all elevations in the powerhouse. Later, after the foundation concrete for the structural steel, speed rings, and scroll cases was complete, permanent bench marks were established. One was near each unit so that it could be seen from a level platform between the units and near enough the same elevation as the crown plate landing of the speed rings to permit using the same micrometer rod. Additional bench marks were set at various locations and elevations to be used for the remainder of the powerhouse superstructure. Closed loops of differential levels were run between these bench marks and bench mark A; loops were balanced out to get as nearly perfect relative elevations as possible and were used for all future work. Bench marks were carried up the steel column with punch marks in much the same manner as the control points.

Lay-out of the speed rings presented the most interesting lay-out problem of the entire job. The remaining equipment in the powerhouse required no unusual lay-out work. Substantial reference points were provided on the east-west and north-south center lines of each unit to establish the vertical center line of each unit. Heavy U-bars made of 1¼-inch square reinforcing steel were placed in recesses in the east and west turbine excavation walls on the center line of each unit at elevation 841.5. Similar U-bars were placed in the downstream wall on the center lines of units Nos. 1 and 2. A steel bar was grouted in the floor of the pipe trench near the south edge of the first stage of construction at elevation 843 on the center line of units Nos. 1 and 2. A control pedestal 6 feet square at elevation 821 and 3 feet square at elevation 840 was erected on the center line of units between the two scroll cases. Steel H-beams were embedded in the north side of the pier to form two platforms or level-stations at such elevations that the draft tube liners could be set from the lower one and the speed rings could be set from the upper one. In the top of the pedestal a heavy U-bar was embedded with the top of the bar at elevation 841.5. Just prior to setting the draft tube liner, the

center lines of both units were carefully marked on U-bars by making a small hack-saw mark. The draft tube liner, and later the speed rings, were assembled and lined from plumb bobs suspended from the intersection of piano wires stretched between the established line points. They were brought to grade by using a level set up on the platform between the scroll cases. This arrangement gave a permanent line, set both the liners and the speed rings, and saved the time of an instrument man during the preliminary setting. Both the liner and speed ring were carefully checked with an instrument before concreting.

An instrument stand, constructed of a section of 15-inch spiral welded pipe with a flange on the lower end which was tack-welded to the spider at the top of the draft tube liner was used for final adjustment of the speed rings. A steel plate adapted to fit an instrument head was welded to the top of the pipe. In leveling the speed ring an accurate 20-second transit was mounted on the top of the spiral pipe, brought to the exact center by sighting the line points and shifting the head, and readings were made at 12 points around the top circumference of the ring.

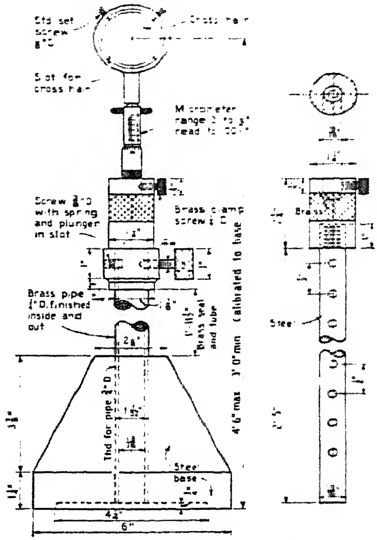


FIGURE 159.—*Micrometer height gage.*

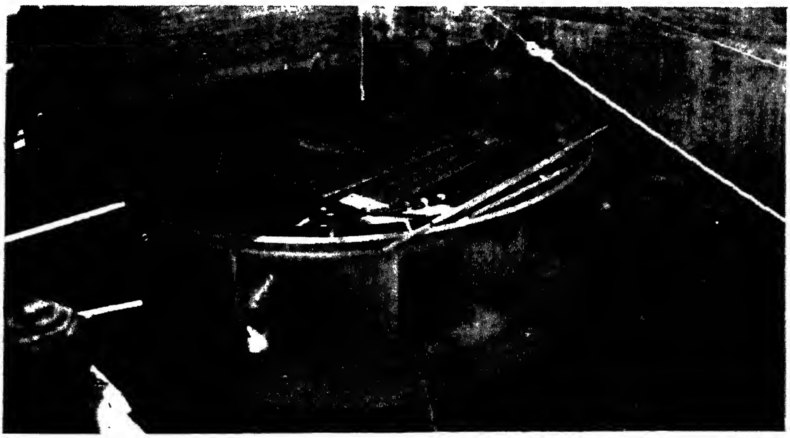


FIGURE 160.—*Checking speed ring with machinist's straightedge.*

These readings were made using the micrometer height gage shown in figure 159. Care was always taken to see that the gage remained a uniform distance from the face of the speed ring and hence a uniform distance from the transit to balance any error between the level bubble and line of sight of the instrument. After a set of 12 readings had been taken, the speed ring was leveled by means of the turnbuckles and leveling jacks. It was found expedient to install a telephone from the instrument platform to the jacks below to direct this adjustment.

After leveling with the transit, check readings were taken with a 10-foot machinist's straightedge and spirit level. An adjustable screw support was made to fit the instrument plate to hold one end of the straightedge, and by leveling this support with the highest point on the ring, any variations were measured with thickness gages. Instrument readings were taken by the Authority's engineers and the straightedge readings by the turbine representatives. The two methods compared very closely and usually the two sets of readings did not differ more than 0.002 inch. Any greater variation was checked again, using both methods. Additional checks on the levels of speed rings were taken from time to time as erection of the turbines and generators progressed to check any possible movement from the original position.

EXCAVATION

Excavation for the dam involved the handling of 183,500 cubic yards of earth and 286,000 cubic yards of rock. This work was divided into three distinct operations: In the first operation, overburden ranging from 15 to 50 feet in thickness was removed from the east bank of the river. The area extended 950 feet along the axis, from the edge of the river to the end of the gravity section of the dam. It was 425 feet wide for the first 200 feet from the edge of the river, the powerhouse, and tailrace sections. In the remaining 750 feet, the width varied from 200 feet at the east end of the powerhouse cut to 75 feet at the east end of the dam.

The second operation involved the removal of all rock necessary to secure a good foundation in the east abutment, powerhouse, tailrace, and spillway areas. The spillway section included an area 375 feet long extending across the width of the normal river channel and 425 feet in an upstream and downstream direction. In the spillway section proper, the cut averaged about 18 feet, and in the east abutment area from 12 to 14 feet, while in the powerhouse area rock was excavated as deep as 30 feet for the draft tubes.

Third, on the steep west or right bank, where practically no overburden existed, the weathered rock had to be removed in such a manner as to deposit the excavated material at the river's edge for handling by shovel into trucks for disposal. This operation was necessary because it was impossible to work on this extremely steep slope with shovels and trucks.

The quarry needs were the ruling factors in the selection of most of the excavating equipment, and in order to utilize the heavy quarry excavation equipment, the excavation of the dam foundation was scheduled well in advance of the opening of the quarry. By this

arrangement, approximately 75 percent of the heavy foundation excavation was completed prior to the opening of the quarry in June 1934. The remainder of the foundation excavation was done by small shovels and rented trucks.

Cofferdam No. 1 area.

The first area in which work was concentrated was protected by cofferdam No. 1, and included the entire east abutment, powerhouse, and three blocks of the spillway. This area contained approximately 68 percent of all dam, powerhouse, and tailrace excavation and lent itself favorably to the use of the heavy equipment. Earth excavation was started in November 1933 on the east bank of the river while the first cofferdam was under construction. Part of the

FIGURE 161.—*Excavation in cofferdam No. 1.*

spoil from this area was used to fill the cofferdam and the remainder was wasted upstream from the dam. By the time the cofferdam had been unwatered in the latter part of January 1934, 56,000 cubic yards of earth had been moved from the area. The equipment was moved immediately into the cofferdam for rock excavation.

Total stage 1 excavation amounted to 376,505 cubic yards of material. Blasted rock was concentrated for the two 3-cubic-yard shovels by tractors equipped with bulldozers. Transportation of the spoil to waste pumps was handled by six 12-cubic-yard Boulder-type body dump trucks, two 8-cubic-yard trucks, and a fleet of from fifteen to twenty-five 4-cubic-yard rented trucks. This excavation was almost completed by June 1934, when the large shovels and TVA-owned trucks were moved to the quarry. Completion of the excavation, which consisted of clean-up and minor excavation items, was done by 1¼-cubic-yard shovels and rented trucks.

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The east bank sloped upward from the river's edge on about a 20-percent grade, with the rock conforming to a series of steps. Rock excavation there consisted of the removal of weathered rock conforming generally to the stratification. As excavation progressed in this area, some rather large clay-filled seams were revealed as forming the floor of several of these steps or benches. Excavation was carried eastward on the bench at elevation 855 as an open cut as far as station 14+45, where it became apparent that above the seam a thick stratum of solid rock existed. It was decided that beyond this point the most feasible procedure would be to drive five tunnels, spaced about 20 feet apart, to a point where the seam had closed sufficiently to be sealed economically with grout. Another tunnel was driven on the bench at elevation 882 and still another on the bench at elevation 906. The longest of these tunnels extended 250 feet into the east abutment. In most cases, the tunnels were driven with the seam at the top in order that the limits might be more effectively determined. Excavation of the tunnels was entirely by hand. The manner in which they were filled with concrete is described on page 368.

Drilling.—Drill holes for permanent structures excavation were comparatively shallow, ranging from 5 to 8 feet deep and spaced about 6 feet on centers. Line drilling was done to the full depth of cut around the limits of areas to be excavated before any blasting was done. This practice was followed:

1. To protect the surrounding rock outside the area of the structure from damage due to blasting in the excavated area.
2. To confine the excavation to the desired area and thus eliminate the cost of handling unnecessary excavated material and the requirement of additional concrete to fill these spaces outside of the desired area.
3. To present a more workmanlike finished structure.
4. Along the stream arms of cofferdams Nos. 1 and 2 to prevent breaking under the cofferdam, which would have endangered the safety of the structure.

In the east abutment area after the overburden had been removed, rock excavation was made with no definite attempt to close line drill to the neat lines of the structure. Drilling was done, however, to conform to the outlines of the structure as nearly as possible. Much lighter blasting was needed as the cut in this area was not so deep as in the stream bed. Also, the presence of the deep blanket of overburden on the rock adjacent to this area lessened the danger of disturbing rock outside the area of blasting. All of the stage 1 drilling was done by wagon drills except near faces which had been line drilled and in cleaning up the bottom of the cuts. In these places jackhammers were used to lessen the possibility of disturbing the adjacent foundation rock.

Blasting.—Dynamite was used exclusively as the explosive for excavation work. Four types of dynamite in two strengths were used: 40 percent gelatine, 40 percent semigelatine, 40 percent ammonia gelatine, and 60 percent gelatine, all of which were supplied in 50-pound cases of 1¼- by 8-inch cartridges. The first two types comprised the bulk of the dynamite used. Electric blasting caps were used exclusively as detonators throughout the entire job. They

were purchased in two strengths: No 6 and No. 8, with 4-, 6-, 8-, 10-, 12-, and 20-foot leads.

Dynamite and detonators were delivered by vendors' trucks to magazines on the job. A magazine for dynamite and another for detonators were located on the west bank of the river about 1/2 mile downstream from the dam. The dynamite magazine had a capacity

FIGURE 162.—*Line-drilled faces.*

of 80,000 pounds, and the contracts with vendors were arranged so that partial deliveries were made as needed, thus assuring a fresh supply of explosives at all times. These magazines supplied only the permanent structures excavation. Two additional magazines, located approximately 1/4 mile south of the main warehouse, served the quarry.

Holes were loaded with from 4 to 6 pounds of dynamite. Usually about 12 to 15 holes were fired at one time and only on very rare occasions did the total dynamite in 1 shot exceed 100 pounds. Shallow holes with light shots were used to allow the foundation rock to remain in an undisturbed condition. Blasts were usually made at shift-changing time.

Every known safety precaution was taken to protect the employees handling explosives and those who worked in the area where explosives were used. As a result, no fatalities and only one serious injury resulted from the use of explosives.

Delays.—Only two delays of consequence occurred while excavation in area No. 1 was under way. The first delay came on March

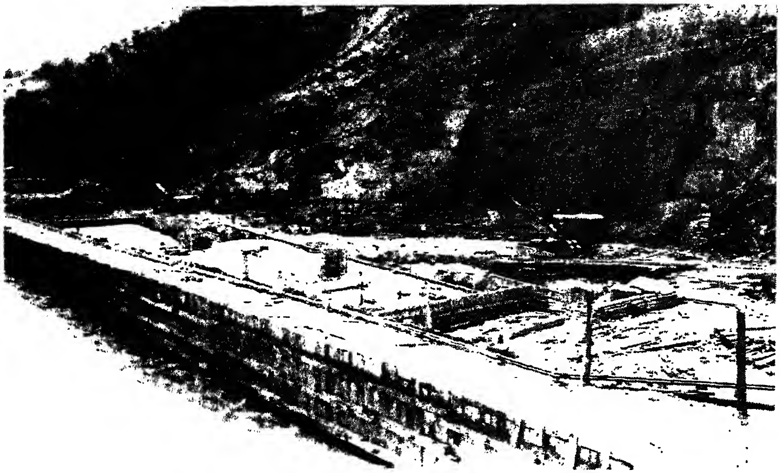

FIGURE 163.—*Excavation in cofferdam No. 2.*

5, 1934, when heavy rainfall in the upper basin resulted in a discharge of 47,000 cubic feet per second at the site, and the cofferdam was allowed to fill as a safety measure against overtopping and possible failure. The cofferdam was pumped out and work was resumed on March 8, 1934. The other delay occurred between May 18 and 23, 1934, pending a decision[12] by the board of consulting engineers as to whether or not excavation should be carried deeper in the spillway area. It was finally decided to excavate 12 to 15 feet deeper to intersect a seam at about elevation 800 in this area from the heel of the dam to 120 feet south of the axis beginning at the middle of block 33 and covering the entire river bottom.

Cofferdam No. 2 area.

Unwatering of cofferdam No. 2 was completed on August 11, 1934. Excavation was started at once and continued until completed about 2 months later. Excavation in this area amounted to 7½ per-

[12] See appendix B.

cent of the total excavation. During the use of the cofferdam 30,862 cubic yards of material were removed, and after the cofferdam had been removed, 10,532 cubic yards of additional material were excavated. Very little overburden existed in this area and work on the west abutment consisted largely of scaling off the weathered rock. Work on the steep west bank was not confined to this stage altogether as it was carried on intermittently during the time when work was being concentrated in the first stage area. The majority of the excavation on the steep west bank was completed along with that inside the cofferdam, and only the final clean-up, all of which was done by hand and wasted upstream, remained to be done as concreting progressed.

FIGURE 164.—*Excavation in cofferdam No. 3.*

Excavation was handled by three 1¼-cubic-yard capacity power shovels. These shovels were not kept continuously in this area because at times it was necessary to use one and sometimes two of them in final clean-up work preparatory to concreting the foundation in the first excavation area. A fleet of from 15 to 25 rented 4-ton trucks was used to carry the material from the shovels. All material removed from the cofferdam was wasted downstream.

Drilling and blasting in this area was done similarly to that in cofferdam No. 1, except that the steepness of the slope precluded the use of wagon drills and made it necessary to use jackhammers for all drilling. The west bank cut presented no difficulties except on one or two benches where partly decomposed rock was encountered. There it was necessary to go to a considerable depth into the hillside to reach sound rock. The worst condition of this type existed in block 46, where it was necessary to carry a bench at approximately elevation 825 into the hill 15 to 18 feet. This bench extended from about 12 feet upstream from the axis to 8 or 10 feet downstream.

Cofferdam No. 3 area.

The third cofferdam enclosed a relatively small area. Excavation was handled by two $1\frac{1}{4}$-cubic-yard shovels and rented trucks. Excavation in the area amounted to 31,734 cubic yards or $5\frac{1}{2}$ percent of the total foundation excavation. Actual work was started on November 5, 1934, and was completed during the latter part of January 1935. Access to the cofferdam was by means of a bridge from the west bank, and all excavation was wasted downstream from the dam in the same general area as used for the deposition of material from cofferdam No. 2. Drilling and blasting procedures were similar to those previously described.

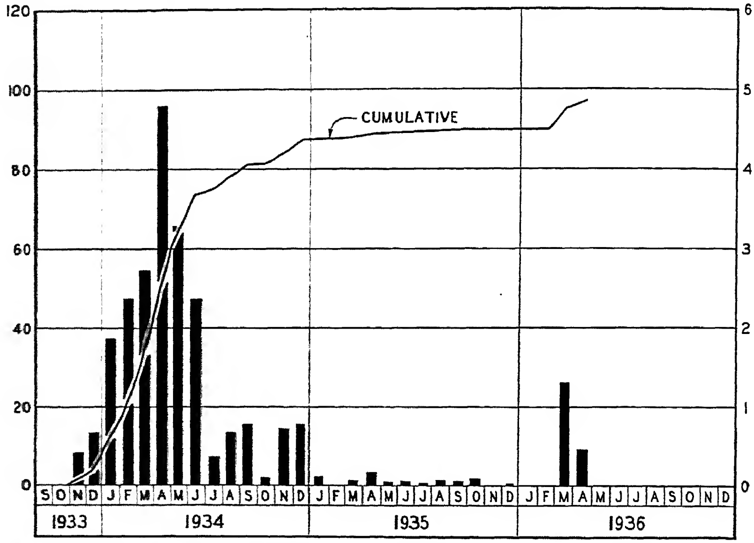

Figure 165.—*Monthly earth and rock excavation for dam and core trench.*

Miscellaneous excavation.

In addition to the excavation included in cofferdams Nos. 1, 2, and 3, work was done in the tailrace area outside of cofferdam No. 1. In this area approximately 34,724 cubic yards of rock were removed. The work was started in March 1936 after one of the large 3-cubic-yard shovels had been released from the quarry. In addition to the 3-cubic-yard shovel, a $1\frac{1}{4}$-cubic-yard shovel was used. The work was completed in April 1936.

Several minor items of excavation were not included in the areas mentioned above. Chief of these were the excavation for the power intake approach channel and final clean-up prior to concreting. An auxiliary cofferdam was built to unwater the area for the power intake channel. This work was carried to completion at the time when the second excavation stage was in progress. The quantity involved was almost negligible, compared to the total excavation for permanent structures.

FOUNDATION TREATMENT [13]

The foundation contained numerous extensive seams, joints, and solution channels. Core borings and geological surveys, made preliminary to the selection of the site, revealed the general character of the formation, and the additional borings that were made for the detailed investigation and grouting of the foundation located the principal seams. From the information obtained, it became evident that an extensive program of treatment would be required in order to seal the foundation against excessive leakage.

Under the dam proper, the treatment was divided into two parts: shallow, low-pressure grouting covering the entire area of the foundation; and deep, high-pressure grouting to form an impermeable curtain under the heel of the structure. On the reservoir rim, the work involved locating those portions in need of treatment to prevent excessive leakage, and the grouting of the areas found to be faulty.

Under the structure, all seams intercepted by the holes for both shallow and deep grouting were carefully washed free of unsound material before grouting was attempted. In the reservoir rim, consolidation rather than replacement of the material in the seams was desired, and washing was not attempted.

The exceptional care that was exercised to prevent the occurrence of leaks through the foundation and the reservoir rim adjoining the abutments of the dam has, as judged by the absence of leakage, well repaid the effort that was expended. The volume of both drilling and grouting was large, and it was necessary to use a variety of methods to cope with the problems that arose during the prosecution of the work.

Early foundation investigations.

Closure of the first cofferdam required some grouting in order to stop leaks through seams into the enclosure and, in drilling to effect this seal, it was learned that both open and clay-filled seams were so numer-

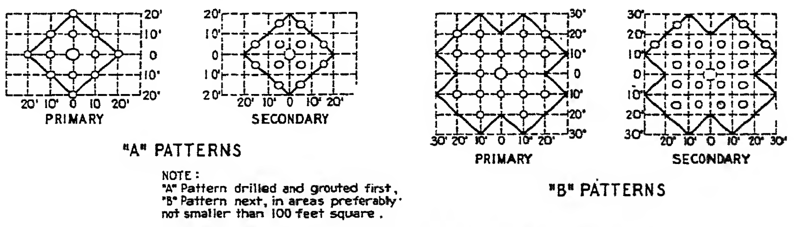

FIGURE 166.—*Patterns used in drilling foundation grout holes.*

ous and extensive that only a systematic plan of treatment would succeed in effectively tightening the foundation. Accordingly, after some study, a plan was evolved whereby the entire foundation was to be drilled on a grid of wagon drill and core drill holes located at intersections of lines 10 feet apart running normal to and parallel with the axis of the dam. Figure 166 indicates the manner in which this grid

[13] This discussion of foundation treatment has been taken from TVA Technical Monograph No. 28, Foundation Treatment at Norris Dam, James S. Lewis, September 1937 (price $1.50).

was divided into interlocking A and B patterns with a $5\frac{1}{2}$-inch core hole at the center of each pattern.

The plan also called for the exploration of the holes to locate all seams and the thorough washing of these seams to remove clay and loose material. As soon as the foundation rock was exposed in the first cofferdam, the drilling of exploratory holes was started. Four lines of 30-foot wagon drill holes, two of which were normal to and two of which were parallel with the axis, were so located as to cover the exposed area of the foundation. In addition, a line of core holes was located 20 feet upstream from and parallel with the axis.

Exploration of drill holes.—It was hoped that the wagon drill operators could satisfactorily determine the occurrence and thickness of seams and report them to a recorder. However, experience soon revealed that this method of locating seams was unsatisfactory. In order to reduce the uncertainty attending the information obtained from the drill operators, an exploring instrument known on the job as a "feeler" was devised for locating seams and measuring their thickness. It was suspended on a calibrated $\frac{1}{4}$-inch wire rope and was equipped with a latch that automatically released the legs when the bottom of the hole was reached. This device consisted of a pair of steel legs so hinged that the weight of the instrument caused them to bear outwardly against the wall of the hole. When a seam was encountered and the wall of the hole no longer acted to restrain the legs, they snapped suddenly and vigorously outward with a force that was easily felt by one handling the explorer on a line from the surface. The thickness of the seam was measured by the free vertical movement that was permitted. This method of locating and measuring seams in wagon drill holes proved satisfactory and, by increasing the length of the legs, the same instrument was used with even better results in vertical $5\frac{1}{2}$-inch core holes.

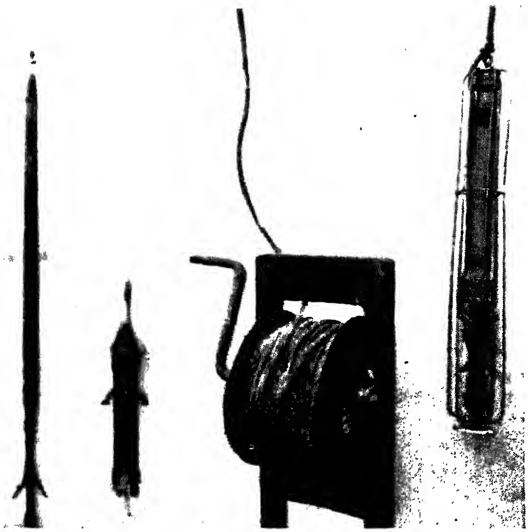

FIGURE 167.—*Types of hole explorers.*

Periscope.—Before the "feeler" method of exploration was accepted as reliable, it was deemed advisable to dispel any doubts as to its effectiveness, and a periscope was designed for checking the results of the exploration of the $5\frac{1}{2}$-inch core holes. This periscope was made of thin-walled brass tubing in sections about 10 feet long,

fastened together by means of a bayonet joint. The length of the barrel was marked off in feet to facilitate reading depths to seams and the unit was handled by suspending it on a rope from a wooden tripod. It was used at a maximum depth of 75 feet, but is not, in the form described, suitable for use under water or in crooked holes. This device was successfully used to test the effectiveness of the explorer, verifying the findings with it in every case, after which it was abandoned in favor of the simpler and faster method of exploration.

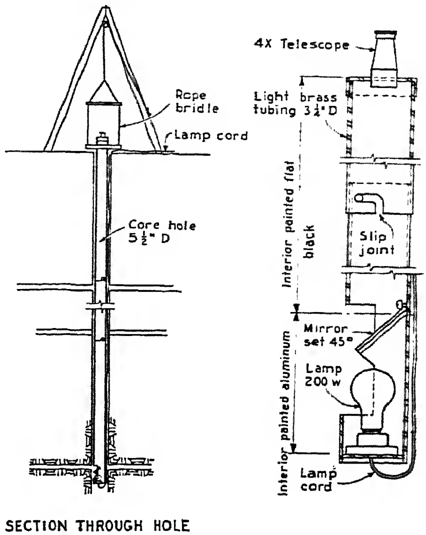

Figure 168.—*Periscope for exploring drill holes.*

Foundation studies.—During the early stages of the excavation, five 5½-inch shot drills and one 36-inch shot drill were placed in operation in the foundation area to secure sufficient data to formulate a definite foundation treatment plan. Exploration holes were drilled varying from 40 to 80 feet in depth. Using the information obtained from all of the eploration holes, sections parallel and normal to the axis of the dam were plotted to show the location of the seams. On these sections, a continuity of gently rolling seams dipping slightly downstream could be discerned, as well as a similarly rolling, almost horizontal, stratification parallel to the axis of the dam. The seams were extensive in area and varied in thickness from mere contact lines of bedding planes to openings of several inches. Some were water bearing, while others were either filled with clay or open. The rock, as evidenced by the cores, was of an excellent quality of hard, close-grained dolomite that tested about 25,000 pounds per square inch in compression. At a depth of about 28 feet below the original rock surface, a layer of badly broken, seamy rock was found to exist generally under the entire area of the foundation. The thickness of this fractured band was approximately 3 feet and the rock was of such quality that no apprehension was felt as to its ability to carry the load imposed by the structure. Since it was felt that this material could be satisfactorily treated by grouting, it was decided that it would be unnecessary to excavate down to it.

Large core holes.—The value of an investigation made by means of small holes is greatly increased when supplemented by holes large enough to permit the entry of a man. This permits the rock to be studied in its original, undisturbed state to an extent that is not possible in shafts that have been excavated with explosives, wedges, and percussion drills. The large cores do not suffer from

grinding and blocking as they do not rotate within the drill bit. This project was the first to use 36-inch core drills for foundation exploration, and it was found that the cost of the 36-inch holes was not excessive. The greatest limitation upon this work is imposed by the ground water. The first 36-inch hole was carried to a depth of 44 feet where it was necessary to abandon it on account of the heavy inflow of groundwater. The depth at which this inflow became so great that the driller was unable to work in the bottom of the hole to remove core usually fixed the depth to which the hole could be drilled. To remove the core when a full barrel (30 to 36 inches) had been cut, the drilling tools were removed and a man lowered to the bottom of the hole. A shallow hole, into which a

FIGURE 169.—*Typical foundation area.*

hoisting pin was wedged, was drilled with a jackhammer in the center of the core and a very light charge of dynamite placed in the groove beside the core. Explosion of the dynamite broke off the core at the base, and it was hoisted to the surface. Considerably greater depth could be reached in any case by advance grouting of the region to be penetrated.

The knowledge obtained from drill records, exploration records, inspection of 36-inch holes, and observations of the structure of exposed rock was correlated to form the basis of the final plan of treatment. It was decided to remove the rock under that part of the gravity section lying in the river bed down to a seam that underlaid the original foundation excavation at a depth of approximately 12 feet. The excavation under the spillway apron and powerhouse was to be carried only sufficiently deep to meet construction requirements or to remove weathered and unsound material. The surface of all of the rock exposed at seams or bedding planes was found to be dipping gently downstream and was marked by frequent domes and hollows that would afford an excellent mechanical bond to resist

sliding. The great frequency of these irregularities is shown clearly in figure 169.

Drilling scheme.—The scheme of interlocking patterns was found to give a better chance of forming a complete grouted cut-off than any simpler method considered. It also possessed the advantage of permitting a large number of holes to be drilled simultaneously, thus contributing toward the improved progress of construction at a stage when delays might prove costly. It was a generally recognized advantage to cover the foundation with concrete as soon as possible in order to reduce the possible damage that might be caused by floods and prevent spalling of the newly exposed rock by temperature changes.

Washing seams for shallow grouting.

It was desirable to remove the clay and other unsound material from seams before grouting, and the methods of washing devised were greatly facilitated by the group system of drilling. The water

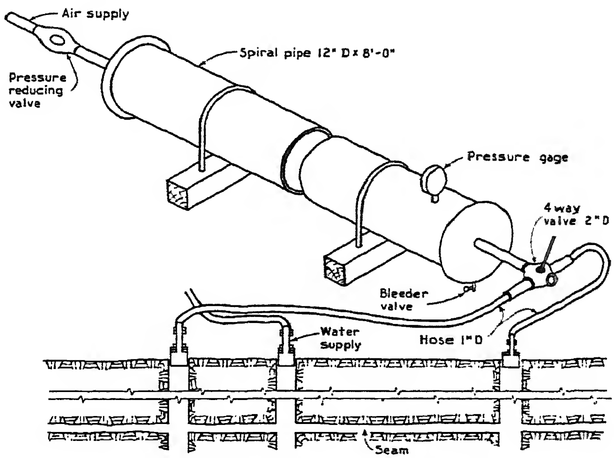

FIGURE 170.—*Flow reversing device.*

emerging from the adjacent holes would bring part of the material to the surface, and it is probable that large quantities of loose material were washed to distant areas in the seams. A mixture of air and water was found to be more effective than water alone as the expansion of the air produced a turbulent flow that increased the erosive powers of the fluid. The air also acted as a lift to bring the material to the surface through other holes.

Flow reversing device.—To facilitate the washing of seams, a flow reversing device was used for reversing the flow of the air. In washing a pattern, it was customary to connect a supply of water to one hole to connect air to the holes on each side of the water connection. By manually operating the four-way valve at frequent intervals, the water in the seams was maintained in a turbulent state that

was very effective in dislodging clay and decomposed rock. Figure 171 shows a group of holes undergoing washing, with the mixture of air and water gushing from a hole near the air receiver in the center of the photograph. When the washing of a pattern was started, most of the holes would be capped and, as the flow became clear, the caps would be switched about to change the path of flow of the fluid. In this manner each pattern was washed until the clarification of the effluent water indicated a reasonable freedom from clay and other undesirable material. These methods were used under the spillway apron and powerhouse.

Washing individual seams.—Under the gravity section, each seam was isolated and washed individually by means of the double expander. After exploration of the hole had disclosed the location of

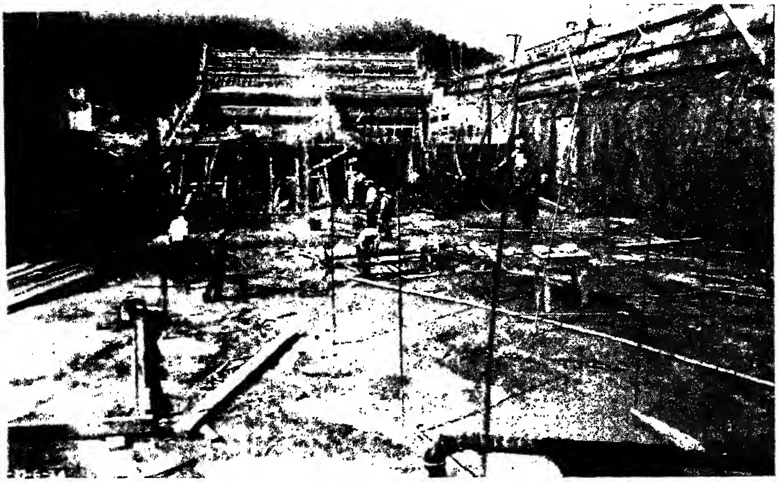

FIGURE 171.—*Washing operations.*

the seams, this expander was inserted in the 5½-inch core hole located at the center of the pattern. The double expander was placed so that the rubbers were an equal distance on each side of the lowest seam. The expanding nut was then tightened and water and air forced in until all possible interconnection with the wagon drill holes in the group had been established. As soon as reasonably clear water flowed from a hole, it was capped in order to force a passage to other holes. Frequently, holes that offered great resistance at the beginning of washing operations would open and take water quite freely after pressure had been applied for several hours. The washing was always started at the lowest seam in a hole, and progressed upward. When the exploration revealed a seam so close to the bottom of the hole that it could not be spanned with the double expander, a single expander that permitted a through passage of the water was set above the seam. Normally, the rubbers on the double expander were spaced 24 inches apart but, when a closely seamed zone exceeding in thickness

the length of the span occurred, the spacing was increased. At the same time that the seams underlying a pattern were being washed individually through the core hole, the air receiver used for producing turbulent flow was connected to the wagon drill holes. Finally, to complete the washing operation, every hole was blown clear of muck with an air jet immediately before grouting was started. The length of time required for working on a pattern before it could be considered sufficiently washed was found to vary between 6 and 24 hours and was determined largely by the judgment of the inspector.

Pressures of air and water for washing shallow grout holes were limited to 30 pounds per square inch, and it was at times necessary to reduce this limit in order to avoid lifting the rock. The safe pressures for washing and grouting were best determined by learning the depth to the uppermost seam. The weight of the rock over this seam was then computed. The theoretical pressure could ordinarily be exceeded somewhat as it was safe to assume that pressures did not exist under the entire area and that the surrounding rock would offer restraint through slab action. Upheaval gages, to give an indication when the safe pressure was being exceeded, were located at intervals over the area of the foundation. These gages were observed during both washing and grouting operations and frequently served as a warning that the safe pressure was being exceeded.

FIGURE 172.—*Double expanders used in 3-inch and 5½-inch holes.*

Shallow grouting.

In general, the system that has been described was used in preparing for the shallow grouting of the foundation area, but at times a modification of these methods was used to adapt them to special conditions. Under the spillway apron, where it was considered important to reduce uplift pressures to a minimum, the A and B patterns were first drilled and grouted to a depth of 20 feet. Following this primary grouting, a system of 40-foot holes, evenly spaced between those first drilled, was superimposed upon the original patterns and the same sequence of operations was repeated.

The pattern system of grouting, as described, was followed in general over the greater portion of the foundation area. Only under the spillway apron and in blocks 25 and 26 was the primary pat-

tern system followed by a secondary system extending to a greater depth. Under the gravity section, grouting was always started from the core hole at the center of the pattern and the wagon drill holes were capped as overflow occurred. After refusal had occurred on the core hole, the wagon drill holes were grouted individually and ordinarily took very little grout. The quantity depended to some extent upon the length of time that had been spent in grouting the core hole.

In the second cofferdam enclosure, the above plan was altered wholly as a matter of expediting the work to avoid delaying the construction program. The alteration affected only the area in this cofferdam lying under the spillway apron and consisted of dividing the area into three parts that were drilled and grouted separately, the holes being located at the intersections of lines forming a 10-foot grid.

The west abutment was composed of a number of narrow ledges underlain by seams, and it appeared desirable to discontinue the pattern system and to treat each seam separately. In the east abutment, the ledges were much wider and the pattern system was modified so as to obtain the best results on each one. These benches resulted from excavating back into the abutments along seams that formed the floor of the excavation, and the seams were usually followed until the ledge rock became sound enough so that it was not economical to remove more. For grouting, drilling from the next bench above was then carried deep enough to penetrate the seam that formed the floor of the bench below. Having drilled out a bench, the holes were flushed heavily with air and water, and 1½-inch pipes about 5 feet long were thrust back into the seam at intervals of approximately 10 feet. The seam between and around the pipes was then packed to a depth of about 18 inches with stiff mortar. When the mortar had hardened, washing was resumed, this time using both the drill holes from above and the pipes that lead into the seam. Grouting was done in the usual manner, capping the pipes when grout flowed from them.

In the case of one particularly large clay-filled seam, it was feared that removal of all of the loose material by washing would so weaken the support of the ledge above that there would be danger of rupturing the rock. To forestall such trouble. several small groups of holes were distributed over the area and drilled through to the seam from the bench above. About one cubic foot of thick mortar was poured into each hole, thus forming supports that would carry the load when the loose material had been removed.

Surface leaks.—When washing indicated that leakage of grout could be expected to occur where seams intersected the surface, these seams were caulked in advance with oakum and lead wool backed up with wooden wedges or packed with a dry mortar of quick-setting cement. Where a seam appeared at the base of a vertical face, a rough form was sometimes built a short distance from the face and filled with concrete through which pipes carried any water that might be flowing from the seam. The pipes were later capped when grout flowed from them. The flow of grout was maintained at all times during the caulking, despite the apparent waste of material, as it was only in this manner that the loss of holes could be prevented.

Holes made useless, usually termed "lost," as the result of a temporary cessation of grout flow had to be replaced by additional holes; and even this did not necessarily insure that the area would be satisfactorily grouted.

It was frequently found that the thin edge resulting where a seam intersected the surface of the rock at a small angle caused trouble by lifting when the seam was grouted under pressure and that caulking of the leak only resulted in additional uplift. In such cases, it was usually necessary to reduce the grouting pressure and pump slowly with a thick mix, allowing some grout to waste until the seam had filled and plugged. When it was learned in advance that such a condition existed, jackhammer holes were drilled vertically from the

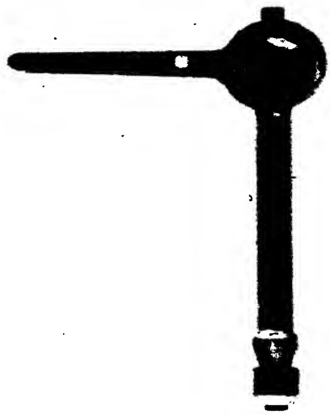

FIGURE 173.—*Expander for wagon drill holes.*

surface just deep enough to penetrate the seam, and they were grouted at a low pressure of 5 or 10 pounds. The deep holes for the patterns were then drilled through this seam and grouted as usual.

Hole connections.—As soon as the drilling of a pattern had been completed, the holes were blown clear of muck to the bottom. After this, a 1½-inch nipple 18 inches long was placed in the top of each hole. If time permitted, these nipples were lightly caulked in with oakum and then grouted with quick-setting cement. It was then necessary to allow the grout to set before washing was started. Ordinarily, time did not permit the use of grout and the nipples were caulked in with lead wool. A quick-working, simple expander was developed after the shallow grouting program was completed, which proved very satisfactory and economical.

Grouting procedure.—It was always considered desirable to place the mixer as near the hole being grouted as practical, but construction activities and the necessity of trucking cement to the location

usually governed the choice. Frequently, it was necessary to pump the grout for a distance of several hundred feet, and a modified circulating system was chosen as offering the most satisfactory and economical method of handling it. As may be seen in figure 143, the grout was pumped to a header connected to the nipple at the hole, where its flow could be controlled by means of valves so that the desired pressure was maintained at all times. This control was necessary when the refusal pressure was determined in advance. The characteristics of a hole were frequently such that, at the rate of pumping maintained, the refusal pressure occurred as soon as grouting was started, although a considerable quantity of grout was taken by the hole before it closed. By keeping the grout circulating at a fairly high velocity in the pipe lines, the tendency of the cement to deposit and form obstructions at fittings was reduced. The velocity required to keep the lines open depended to a great extent upon the prevailing temperature. Also, the modified circulating system possessed advantages when gradual refusal was occurring, as it gave flexibility in controlling the pressure regardless of the rate at which the hole was taking grout.

The value of a pump for handling the grout was much greater than compressed air. The pump afforded additional mixing of the fluid and made possible a flexibility of control and continuity of flow that was not obtainable with the pneumatic grout machine.

Tunnels to seal large seams.

In addition to the grouting of the seams in the abutments, other measures were taken to reduce percolation to a minimum. As the excavation proceeded, tunnels were driven parallel to the axis, following the seams forming the floors of the various benches.

Filling tunnels.—To fill the tunnels in the east abutment, 36-inch core holes were drilled from the surface above and concrete was dumped into the holes in three-cubic-yard batches. The long tunnel under the core wall was filled through an 8-inch pipe from the surface. In the case of the tunnels that were filled through 36-inch holes, the portals were covered by bulkheads that extended several feet above the top of the tunnel. It was the practice to pour concrete into the 36-inch hole until overflow occurred at the bulkhead, thus assuring that sufficient pressure existed to fill the tunnel to the roof for its full length. The impact of the 3-cubic-yard batches dumped from the surface was sufficient to force the mass in the tunnel to travel the desired distance.

In order to permit the injection of grout into any space resulting from the shrinkage of the concrete in the tunnels, and also to serve as air vents while placing the concrete, two 2-inch pipe headers were placed along the full length of each tunnel and 1-inch pipes connected to the headers were extended upward into jackhammer holes drilled 12 inches vertically into the roof at high points in the tunnel. In addition to the pipes inserted in these holes, others were extended into the seams. These were spaced about 10 feet apart and were installed for the purpose of filling the seam between the tunnels. They were connected to the same header, and dry mortar was packed between and around them where they entered the seam. The arrangement of the grout pipes in one of the tunnels is shown by

figure 174. After filling the tunnels with concrete, grouting was always deferred as long as possible in order to permit grouting after most of the shrinkage had occurred. After the tunnels had been filled with concrete and grouted, the areas lying between parallel tunnels were drilled from above in the usual manner and any remaining open spaces were washed and grouted through the holes. Eight tunnels were driven into the east abutment, not including the long one under the core wall, and four were driven into the west abutment, including one that followed a cave.

FIGURE 174.—*Tunnel, with grout piping system installed, ready for concreting.*

Investigations for deep foundation treatment.

The concreting of the structure had progressed sufficiently by the time the shallow grouting had been completed so that the deep drilling for the curtain grouting from within the gallery could be started. For this purpose 5½-inch shot core holes located on the upstream side of the gallery and sloping upstream at an angle of 10° with the vertical were drilled on 10-foot centers for the length of the dam. Vertical upheaval gage holes spaced 60 feet apart and carried to a depth 10 feet in excess of that tentatively chosen for the grout holes were drilled in advances of the grout holes. By drilling these holes so early it was possible to obtain detailed information concerning the location of seams. It was not possible to drill the full length of the dam at once, by reason of the varying stages of construction at different points, but when a section was made available for drilling, these vertical gage holes were always completed first. By combining the logs of the drilling and the exploration records of these holes it was possible, before drilling for grouting was started, to plot a very useful section through the rock underlying the dam, showing the location, thickness, and general characteristics of all the major

seams in this zone. This section, when developed for the full length of the dam, was used as a basis for planning the program of curtain grouting. As additional information was secured by drilling the grout holes, the program was modified to meet new conditions.

Grouping of holes.—The work of drilling and grouting the holes 10 feet apart in the galleries was divided into three parts. First, groups of three holes on 100-foot centers between groups were drilled, washed, and grouted, after which groups of three holes halfway between the first groups were similarly treated. This left space for two holes between the first and second groups and these were drilled last. By following this plan, an effect somewhat comparable to stage grouting was obtained, without the delays incidental to that method. The concentrated washing of the seams, allowed by grouping the holes to secure interconnection between them, was of considerable advantage. The washing of the first groups tended, of course, to remove some unsound material from the surrounding area and the grouting of these holes effected partial consolidation of the area outside of the group. The treatment through the second groups extended to and possibly overlapped the consolidation effected from the first, and the final washing and grouting through the two-hole groups in the partially consolidated areas was designed to complete the formation of the grouted curtain.

Exploring deep holes.—For exploring the 5½-inch inclined holes in the gallery, two devices, as shown by figure 167 were developed. These were essentially improvements upon the original "feeler" used for the shallow work. One was mechanical in principle and depended upon rollers to keep it centered in the sloping holes, while the other was electrical and depended upon a flexible wire cage to keep it centered. Both were very effective in locating seams, but the electrical type was favored by reason of the ease of handling afforded by its lesser weight and the fact that it was probably superior at the greater depths. This device consisted of a centering cage containing a pair of spring-actuated legs so adjusted that an electrical circuit was closed, flashing a light for the observer, when either leg moved outside of the travel limits fixed by a hole of normal size. The cage was suspended on a ⅛-inch bronze cable marked at 5-foot intervals to which was closely taped an insulated single conductor. The bronze cable served as a second conductor. A latch held the legs in the closed position until the bottom of the hole was reached, when they were automatically released and were free to open into any seam encountered as the device was pulled upward. Each hole was explored twice to reduce the risk of missing seams.

Washing through deep holes.

Following the location of the seams in any group of holes, each seam was washed individually by means of the pneumatic expander. This expander consisted of two sleeves of heavy gum rubber separated by a piece of perforated pipe and suspended from the surface on a marked steel cable which was paralleled by the hose carrying the washing fluid. A separate hose supplied air to expand the rubber sleeves and both hoses were lashed to the cable. By centering the perforated pipe on a seam and expanding the sleeves, the whole supply of washing fluid could be directed into the one seam. This pneu-

matic type of expander possessed decided advantages in places where the head room was limited, as in the grouting galleries, or where the holes were very deep. Where the head room was limited, the pipe that supported the mechanical type was made up in short sections making it rather awkward and slow to handle; and, as the holes became deeper, the increased weight further increased the difficulties. A detail of importance on these expanders was the cup, or calyx, that was mounted on top to catch fragments of rock that fall from seams and caving zones. These fragments, if not caught, would wedge in the space between the expander and the wall of the hole and occasionally result in the loss of the equipment.

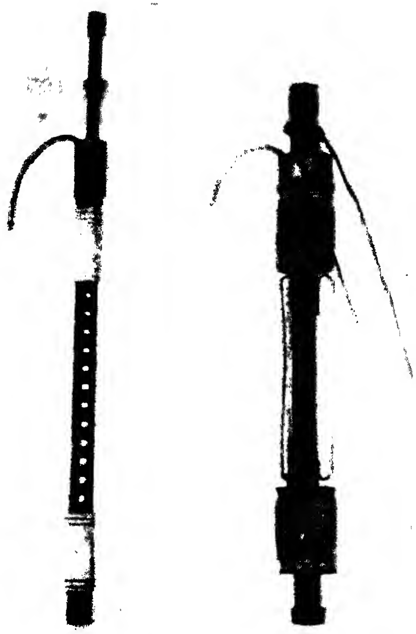

FIGURE 175.—*Pneumatic expanders, for washing 3-inch and 5½-inch holes.*

In washing a group of holes the lowest seam was treated first. The initial quantity of water that the seam would take at the washing pressure was then measured by passing it through a calibrated orifice assembly that contained orifices of different sizes to cover a wide range of flow. Checks were made at regular intervals during washing to learn whether the seam was opening up as the result of the removal of material. Air and water were used, mixed and separately, to loosen the material, and the expansion of the air brought large quantities of muck to the surface from the connected holes. The length of time spent on each seam was judged by the change in the color of the water overflowing from connected holes. If there was no overflow from other holes, the rate of consumption of the wash water was measured until no increase was noted, when the expander was moved to the next seam above. As the overflow from any connected hole became clear, the hole was plugged at the surface in order to force a passage to the other holes in the group, and the operation was then continued. In this manner, each seam in each hole was thoroughly washed. After the washing was completed, and just before the grouting was started, the holes were blown out with air until they were clear of muck to the bottom.

Great stress was always placed upon the importance of thorough washing of the seams as it was felt that the effectiveness of the grouting dependent almost wholly upon the success achieved in removing

the unsound material. The pressure of the wash water at the surface did not, in general, exceed 100 pounds per square inch.

Water-cement ratio.—The water-cement ratio of the grout was determined in advance for each group or for any individual holes not connected with others by measuring the quantity of water which the holes would take at the washing pressure. For very tight holes the ratio was 3.0; for holes offering a fair amount of resistance, 1.5; and for holes that were very open, 0.66. What was termed the normal mix had a water-cement ratio of 1.0. This normal mix was used on holes that were about average in all respects, and it possessed sufficient fluidity to be satisfactorily handled by the pumps. When the ratio was reduced below 0.66, the consistency of the grout was such that trouble was experienced in handling it with the pumps. Extremely accurate control of the water was not considered necessary in view of the fact that much of the grout was pumped into water-filled seams.

Curtain grouting.

After a group of holes had been washed and blown free of muck as described, short single expanders shown in figures 172 and 176 were placed in each hole. If connection with another group had developed during the washing, expanders were also placed in the holes of the other group. When a free connection between groups existed, every effort was made to grout them simultaneously with separate pumps. However, this was not always practical and, in such cases, the lines were extended from one pump to both groups. Pumping would be started on one hole of a group at a rate of between 100 and 200 cubic feet of cement per hour. When grout flowed from the other holes of the group, the overflow was stopped by means of valves installed for that purpose, and pumping was continued until overflow from the connected group resulted. The latter group was then closed off, and the pressure on both groups was measured with the pump running. If the connected group showed appreciably less pressure than the group into which grout was being pumped, the flow of grout was diverted to the former and, from that time to refusal, the flow was changed back and forth between the two groups to maintain approximately the same pressure on both. When overflow failed to develop from a group

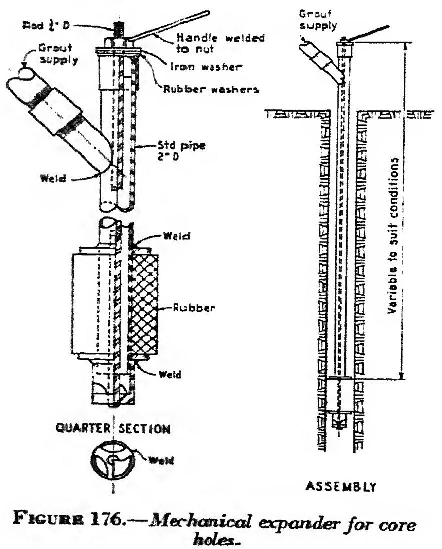

FIGURE 176.—*Mechanical expander for core holes.*

with which interconnection was known to exist, the holes in this group were sampled at intervals by sounding to learn if grout was leaking in. If the soundings indicated that it was coming in at a very low rate, the hole was kept clear by blowing it out until a pump could be connected to it. On holes that offered initial resistance to the inflow of grout, the pumping was maintained at as high a rate as was consistent with the limits of the pump and mixer and the back pressure on the hole, as it was believed that a superior quality of grout resulted when it could be put in under pressure. The pressure possibly had some effect in squeezing the excess water from the deposited grout, thus improving its quality.

Pumping rate.—On holes that offered no initial resistance to the inflow of grout, it was necessary to exercise judgment in determining the rate of pumping. In grouting seams of wide extent, pumping at a high rate might have resulted in forcing large quantities of grout into remote regions where it would have been wasted insofar as the objective was concerned. A better effect was obtained by the judicious use of a much smaller quantity of thick grout pumped slowly. In general, when starting to grout a group of holes, it was better to grout first those that offered the least resistance to flow, as determined by observations made while washing. A refusal pressure of 150 pounds per square inch was required for the curtain grouting, although it was occasionally necessary to reduce this pressure to prevent harmful uplift of the structure.

Upheaval gages.—As a result of the great area covered by the seams underlying the foundation, the danger of raising the structure by the use of excessive pressures was serious, and upheaval gages were installed in the grouting gallery at 60-foot intervals for the length of the dam. These gages as shown in figure 29 consisted of a piece of 1-inch pipe with its lower end anchored by grout in the bottom of a vertical hole that had been drilled 10 feet deeper than the nearby grout holes. Above the anchorage, the pipe was encased in 2-inch, asphalt-dipped, fibre conduit. After the pipe had been anchored by pouring grout through it, it was held in a vertical position by maintaining a strain on the top while the hole was filled with lean, coarse mortar. The strain was held until the mortar had set, thus assuring that the pipe would be restrained against excessive deflection as the result of carrying its own weight as a column. Across the top of the pipe, a bridge of 1¼-inch square reinforcing steel carried a bronze tip that was set approximately 0.030 inch from a similar bronze tip on the pipe. The gap between the tips was measured with a thickness gage at frequent intervals during grouting and, if a progressive increase of as much as 0.010 inch occurred, it was construed as a warning that the safe pressure was being exceeded. When this occurred, the header was removed from the hole and the grout allowed to flow out, the uplift gage being observed constantly to ascertain when settlement of the structure had ceased. When the original position of the structure had been assumed, or settlement had ceased, grouting was resumed at a reduced pressure.

Action of grout.—From observations made on numerous cores drilled from grouted seams and grouted holes it appeared that in seams the solid grout built up slowly in layers as the cement was deposited. It is probable that this deposition began when the veloc-

ity of the grout was reduced as it spread out after leaving the drill hole. As continued deposition resulted in constriction of the passage, the cement was deposited farther and farther from the hole. Chemical affinity between the particles of cement was also a very important factor in this building up of the solid material. Evidence of this fact was seen in cores removed from grouted holes, where it was frequently observed that the cement had built up in concentric layers,

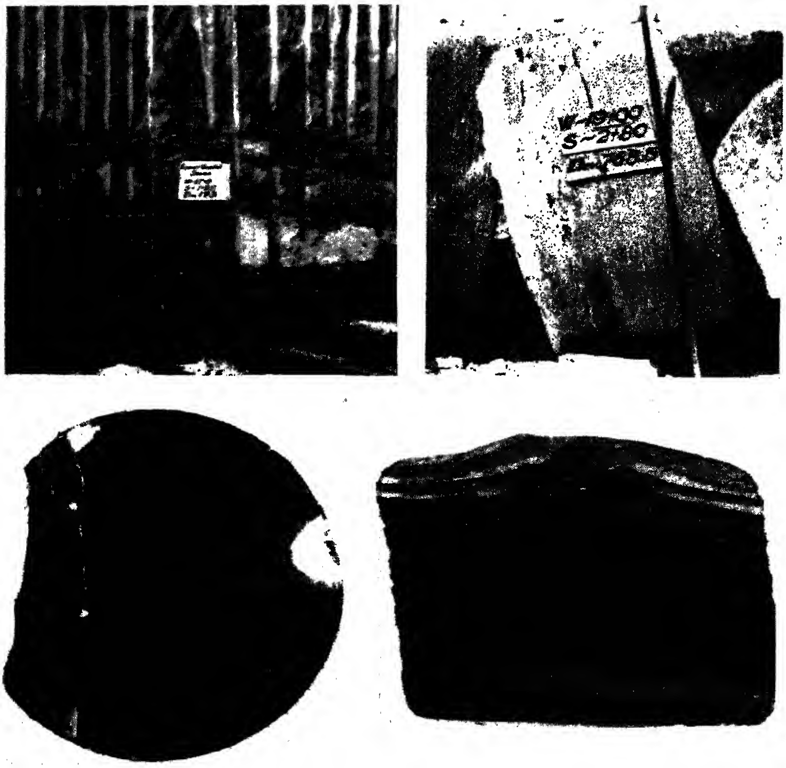

Figure 177.—*Characteristic deposits of grout.*

gradually closing in and reducing the size of the passage. The final closure probably resulted from the tendency of the cement particles in suspension to cohere to the deposited cement, gradually closing off the passageway. Many things may have happened to make the cause of this final closure obscure and uncertain. Samples cored from both seams and grouted holes that seem to substantiate this theory are shown by figure 177. Since the final passage through which the grout must pass was very small, the use of screened cement would doubtless have delayed the closure and increased the

amount of cement that could be injected under any circumstances. Screening would not ordinarily be economically justified.

Leakage.—Careful observations made regularly since the reservoir has been filled have failed to disclose any evidence of leakage. With a full reservoir and all gates closed, the total flow measured in the stream bed 1 mile below the dam was approximately 2 cubic feet per second. This quantity included the flow from a number of springs below the dam that existed before the dam was built. Test holes drilled to penetrate the grouted curtain at intervals of approximately 100 feet throughout the length of the dam have revealed a degree of tightness that was unexpected, the total flow from all holes being approximately 12 gallons per minute. Nineteen drain holes, drilled in the spillway apron to a depth 15 feet below the grouted zone, discharge a total of 0.55 cubic feet per second.

Sealing of rock under core wall.

From the east end of the gravity section of the dam, a reinforced concrete core wall was extended 584 feet through the rolled earth fill and overburden. The scheme of grouting along this core wall was similar to the methods used under the dam. Holes were drilled 10 feet apart to a sufficient depth to penetrate the water table by 10 feet, and at 50-foot intervals the depth was increased to penetrate the lowest known seam. The line of holes was offset upstream 7.5 feet from the center of the wall in order to avoid interference with the excavation of the core trench. As a matter of additional precaution, jackhammer holes 8 feet deep and sloping upstream were drilled 5 feet apart along the bottom of the trench. These jackhammer holes were grouted at low pressure to fill any seams near the surface of the rock in the space between the core wall and the line of deep core holes 7.5 feet upstream. It was decided to divide the grouting into two zones: one above the seam at elevation 965, and one below it. The first drilling was stopped when the holes penetrated the seam. Before grouting, thick dry mortar was poured into the holes to form a plug at the bottom that would prevent the passage of grout to the seam. The seams in the upper zone were washed and grouted to refusal at a pressure of 75 pounds per square inch.

Packers.—As the normal reservoir level was at elevation 1020, it was unnecessary to grout seams above this level, and packers of two types were used to control the limits of the grouting. The elevation of the overburden from which the drilling was done varied from 1,060 to 1,170 and the elevation of the surface of the rock under this overburden varied from 965 to 1,085. The expanding, tapered-body, rubber-sleeve type of packer was developed for use in diamond core holes which are more uniform in size than shot core holes. It was at times necessary to use the cup-leather type when the hole was too large to permit the rubber to jam against the wall of the shot core hole. Both were very satisfactory though time was occasionally lost through having to wait for the cement to set when the cup-leather type was used. When the hole had been grouted above elevation 965, a minimum of 24 hours was allowed to pass before drilling was resumed to carry the hole to the final depth. Following the

exploration and washing of the lower part of the hole, a packer was set immediately below the seam found at elevation 965 and the final grouting was completed at a pressure of 150 pounds per square inch.

Reservoir rim investigation.

The geological survey, made before the site was selected, revealed that the rim of the reservoir for a distance of 9 miles from the east abutment and for 5 miles from the west abutment would, by reason of the geological characteristics of the formation, probably allow some leakage of water from the reservoir. The cost of treating, or even of thoroughly investigating such a lengthy stretch of rim would have been very high, and it was decided to confine the treatment to the narrow and more important portions adjacent to the abutments.

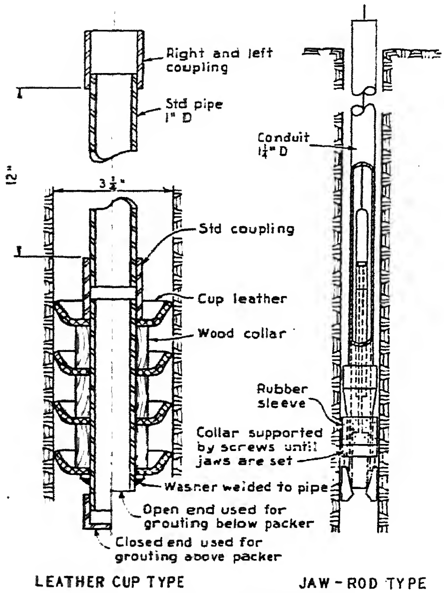

FIGURE 178.—*Core hole packers.*

At the beginning of the rim investigation and treatment, the elevation of the water table underlying the areas in question was used as the criterion by which the permeability of the ridge was judged. That is, a high water table was interpreted as indicating that the rock beneath must necessarily be tight in order to support the water and that, conversely, the rock above a low table was sufficiently permeable to allow drainage of the ground water. Care was taken to make certain that the investigations were sufficiently thorough to avoid mistaking a perched table for the real water table. The first step, after determining the general areas to be investigated, was to drill diamond core holes approximately 200 feet apart on a line along the top of the ridge and to a depth well below the bed of the river. Six holes were drilled on the east side and five on the west side. These holes were sounded daily for a sufficient length of time to determine the elevation of the water table and its relation, if any, to the water in the river. Where apparently open or pervious areas were found, as judged by the sensitivity of the table to changes in the elevation of the river water, other holes were drilled on each side of the ridge to establish a gradient and to ascertain that no tight barrier existed in the side of the ridge away from the reservoir. When the areas had been drilled, daily soundings were made, extending over a period during which sufficient variation in river level occurred, so that the characteristics of the holes were definitely estab-

lished. Some holes followed closely from the beginning every change in the elevation of the river water. Other holes never reacted to changes in the elevation of the river water and some did not begin to fluctuate with the reservoir until it rose to higher elevations, indicating the certainty that an open seam existed at the elevation at which the activity started. From the information obtained by sounding the holes, a contour map of the water table underlying the areas adjoining the abutments on each side of the river was made and this map, in conjunction with contour maps of the ledge rock, and of the ground surface, was used to determine the location of the line of grout holes. Figure 179 shows an arrangement of cellophane straws, containing adjustable solid straws, which was very useful in

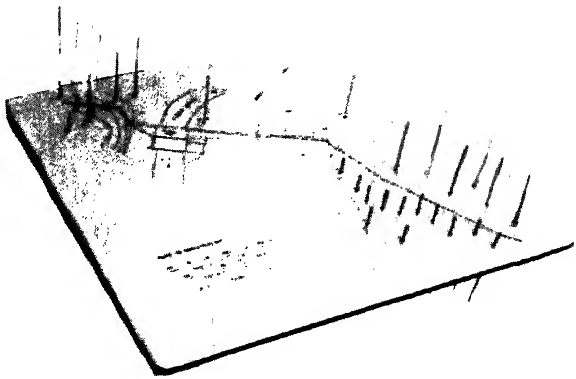

FIGURE 179.—*Model used to study the effect of grouting on the ground water table.*

studying the fluctuation of ground water in the reservoir rim. The line of grout holes is shown on the contour map and the effects of the grouting were plainly discernible when the straws were properly adjusted. On the west side, the line was located well upstream from the extended axis of the dam in an effort to avoid wasting grout into a large cave found during the excavation of the abutment.

Cave in west abutment.—The cave found in the west abutment at elevation 980 was partially filled with clay and crossed the axis of the dam at an angle of approximately 18°, the direction of principal jointing of the dolomite. An effort to follow it by removing the clay resulted in a serious cave-in of the overburden. That part of the cave that had been cleared was filled and a sink was formed at elevation 1,160 under the cableway head tower tracks. This necessitated a cessation of practically all construction activity until the sink could be spanned with a reinforced concrete girder. The sink formed by the cave-in apparently occurred in a large vertical drain or solution channel that had formed through the rock and originally drained the higher elevations to the cave opening on the bluff. Further removal of loose material to permit exploration of the cave was

abandoned as it was feared that additional trouble might be experienced at the head tower tracks.

Sounding holes.—The sounding of $2\frac{1}{16}$-inch holes 400 feet in depth to determine the elevation of the ground water was something of a problem and it was found that the most satisfactory device for this purpose consisted of an inverted cup suspended on a steel tape. A distinct plopping noise resulted when the cup hit the surface of the water. This sounder was suspended on a 500-foot, narrow, graduated steel tape and, when wound on a reel, proved an effective and easily portable device.

Rim grouting.

Having determined the approximate location of the water table and of the rock surface in the areas in question, and having obtained a general idea as to the frequency, location, and size of seams, it was next necessary to locate the lines upon which the grout holes were to be drilled. On the east side, elevation 1,020 was set as the upper limit of the grouting, with holes spaced 50 feet apart. On the west side, due to the cavernous rock, the proximity to the abutment of the region of low water table, and the certain existence of extensive seams, the upper limit of the grouting was raised to elevation 1,050 through the most permeable portion, and the spacing of the holes was decreased to 20 feet. On both sides of the river, the holes were drilled to a depth below river bed.

Jaw packers.—Since practically all of the holes were drilled from an elevation well above the upper limit of the grouting, it was desirable to avoid waste of the grout by confining it to the regions where it was needed. For this purpose the jaw packer was developed. In practice, the packer was set by lowering it to position on $1\frac{1}{4}$-inch electrical conduit. Conduit was chosen in preference to pipe because the short joints of uniform length and lighter weight were easier to handle. Having placed the packer at the desired elevation, a weight called a jar rod was lowered on a line until it rested upon the tapered jaws of the packer. This jar rod was made with a sliding body so that it could be driven quite forcefully against the jaws by raising and releasing the body with the handling line. The teeth on the packer jaws were forced to grip the wall of the hole and were held in this position by the weight of the jar rod. The conduit was then released slowly and its downward motion forced the rubber sleeve to expand upon the tapered body of the packer until it jammed against the wall of the hole and formed a seal. The jar rod was then withdrawn, as the friction of the rubber against the side of the hole was usually sufficient to support the column of pipe. When the friction was not sufficient, the weight was supported by a clamp that rested upon the top of the casing. On completion of grouting, the packer was removed from the hole and was ready again for immediate use. To preclude the possibility of losing the conduit should the packer become hung in any manner, the connection where it joined the packer was threaded left hand so that the full length of conduit could be released by turning it clockwise.

Water tests.—Having set the packer at the desired elevation in a hole, a water test was made in an effort to gain some idea as to the

quantity of grout that would be taken. The usual range of test pressures varied between 25 and 50 pounds per square inch. This test was of value in that it was generally true that a tight hole would take little grout whereas a very open hole could be expected to take large quantities. Attempts were made to test the holes in 5-foot intervals by using two groups of cup leathers, facing each other and separated by 5 feet of perforated pipe. Although this method had been used successfully, it was found to be wholly unsatisfactory as attempted here. The leathers would catch in seams and bend back,

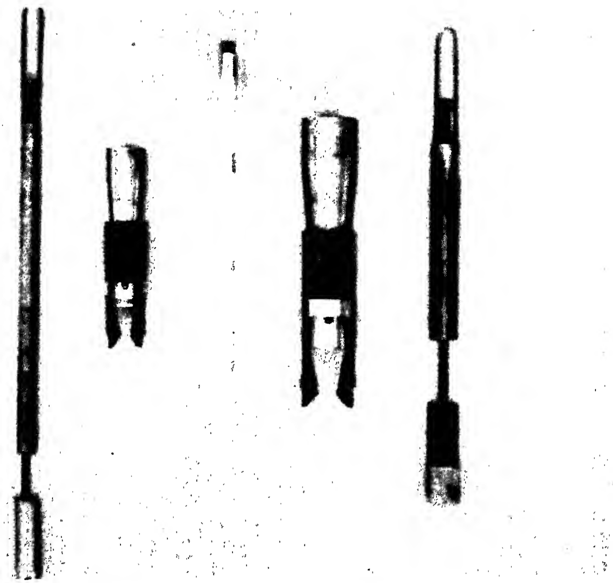

FIGURE 180.—*Jar rods and packers.*

or would reverse from the pressure of the water, or would be placed opposite a seam and have no support from the wall of the hole. The process was too laborious, considering the number of withdrawals necessary, and too uncertain to justify the delay and expense involved.

Seams.—As previously stated, the elevation of the water table was considered very important in determining the locations of the permeable portions of the rim, but, as more information concerning the location and size of the various seams was obtained from the drilling of the grout holes, the work became more and more a problem of following and checking these seams carefully, regardless of the position of the water table. The grouted curtain was ended on each side where the water table was found well above reservoir level. Although it was recognized that it was quite possible that leakage would occur through the rim both within and beyond the grouted portions, it was not felt that the expense of investigating and treating the

entire part of the length that was questionable would be justified. In view of this fact, it was decided that the necessity for any additional treatment would be determined after the reservoir had been filled for some time and the seriousness and extent of any leaks observed.

The grouting of the reservoir rim differed from that of the dam foundations in several major respects. No attempt was made to wash unsound material from the seams, as it was felt that the small size of the $2\frac{5}{16}$-inch holes, spaced so widely, precluded the possibility of applying a sufficient volume of water to do any real good. A complete replacement of unsound material with grout was not sought but rather a consolidation of this material by penetrating and filling the interstices with hard grout injected under sufficient pressure to compact the loose material in the seams. It was known that the openings in the rim were of such size and extent that, by maintaining a high rate of pumping, the grout could be forced to travel unneeded distances, increasing the consumption of material considerably. Accordingly, the rate of pumping was usually limited so that not more than 80 to 100 cubic feet per hour of solid material was handled by one pump. The determination of this rate was not based upon any assumed conditions as to the most effective rate for securing the desired consolidation, but was chosen because the pump and mixer handled this quantity without undue wear and because sufficient velocity of the fluid was maintained to prevent the troublesome deposition of grout in the lines and in the pump discharge chamber.

Pressures.—In general, a refusal pressure was arbitrarily set at 25 pounds per square inch at the surface in the belief that this pressure would afford the desired consolidation in the upper seams without causing undue waste in the lower regions. Since a general tightening of the rim was evidenced as the program neared completion, the pressure was somtimes raised to 100 pounds per square inch in order to consolidate more effectively areas offering greater resistance to the injection of grout.

Use of rock flour.—With the thought that a fine inert material might be used with cement to form a satisfactory and economical grout for rim tightening, a settling basin was built to collect rock flour from the sand classifier of the aggregate plant. This material also possessed value as a medium for improving soils for agricultural use, for which purpose it could have served if it had been found to be unsuitable for grouting.

Preliminary tests.—To determine the suitability of the rock flour cement mixture for grouting, preliminary field tests were made in an attempt to learn the characteristics of the resulting product. The first test was in the form of a trial run of the material through a pipe about 200 feet long, laid with the usual number of fittings and irregularities, and then returning to discharge into the mixer. The grout consisted of equal parts of cement and rock flour and had a water-solid ratio of 1.0. The mixture was circulated from the pump very slowly to give it every opportunity to deposit and build up in the pipe line if there should be any tendency for this to occur. The batch in circulation was replaced with a fresh one every 2 hours to avoid the loss of chemical properties that occurs when cement remains in an agitated solution too long. Samples of the grout were

obtained from the mixer and observed for rate of setting, hardness, and for a general comparison with cement grout. The test was continued for 48 hours, after which time the pipes and pump were dismantled and found to be bright and clean, altogether different in appearance from what would have been expected if only cement had been used. In the case of cement grout, pumped at the same rate, the lines would have plugged with the material soon after the test was started. The samples that had been taken from the mixer were observed to set and harden at a rate that indicated considerable retardation when compared with regular cement grout. These preliminary and decidedly crude tests indicated that the characteristics of rock flour cement grout differed from those of cement grout to a degree that would make it desirable to revise the scheme of grouting if its use was adopted, and established the necessity for laboratory tests supplemented by additional investigations in the field.

Importance of time-of-set.—It may be safely assumed that, in grouting extensively seamed rock at a fixed rate of pumping, the area that will be covered is dependent to a large extent upon the time-of-set of the cementing material. By the use of a slow-setting material, the distance travelled by the fluid may be increased so that areas completely outside of the region requiring treatment will be grouted and the quantity of material necessary to effect consolidation increased appreciably. Obviously, though the unit cost of the grout might be reduced by the use of a cheap, inert material to replace part of the cement, the total cost of treating a given area might be greatly increased as the result of the larger total quantity of material consumed.

Addition of calcium chloride.—For these reasons, tests were made to determine the effect upon setting time secured by the addition of calcium chloride to the grout. It was learned that, by the addition of 3 pounds of calcium chloride to each 100 pounds of the cement, the set could be accelerated to a degree that would counteract the retardation resulting from the use of rock flour to a marked extent. The resulting product possessed characteristics of pumping and handling similar to those of the regular portland cement grout. Compressive tests on specimens removed from the mixer in the field seemed to indicate that the product was sufficiently sound to resist erosion. In appearance, the product was little different from the regular grout, being dense and apparently impermeable, though not quite as hard. Specimens cored from grout-filled seams, after the rim grouting was under way, failed in compression at 2,000 pounds per square inch at an approximate age of 45 days.

Handling rock flour.—It was found after a trial run that the rock flour contained a quantity of fine clay that failed to disintegrate in the course of mixing in the regular manner. It also contained a small proportion of coarse material that made screening desirable. After some experimentation in an effort to disintegrate the rock flour with jets, it was found that the most satisfactory method of breaking it down was by means of a separate, mechanically agitated mixer from which the rock flour passed in solution, through a screen, to the grout mixer. The mixers were very similar to those used for grout, being driven by the same type of air motor but having one compartment instead of two. For proportioning, the rock flour was

measured by counting the number of shovelsfull placed in the mixer, with an occasional check with a measuring box. Though this method seems crude, the nature of the material, its cohesiveness and resistance to breaking down into a granular form, made any other practical method very slow and difficult.

Sand for grouting.—During the preliminary grouting for sealing the first cofferdam, one attempt was made to obtain economy by mixing sand with the grout. The sand was placed in the mixer with the intention of handling it in the regular manner, and in a very short while, the pump and lines were plugged solidly. After this unsatisfactory trial, the use of sand was not attempted again until it became necessary to grout some holes in the rim that were known to penetrate very large openings. In this case, the holes took grout so freely that a vacuum existed at the surface and a funnel was installed on the header at the hole. Sand was shoveled into the funnel where small water jets aided its flow. Knowing the rate at which the grout was being pumped, it was possible to proportion the mix by adding sand at a fixed rate. In this manner, the sand was handled very satisfactorily and a large quantity of it was placed in the two holes in which it was used. However, the sudden and peculiar manner in which the holes refused the grout raised some question as to the wisdom of using this material, and its use was discontinued. Subsequent redrilling and regrouting of the holes that had refused to take more of the sand grout further confirmed the belief that it was unsuitable, as it was found that it was possible to pump a large quantity of the cement rock flour grout into the same seam that had been previously

FIGURE 181.—*Sand grout cores.*

plugged with the sand mixture. Cores of the sand grout that were later removed from nearby holes indicated a tendency toward segregation, the material being lean and crumbly and generally poor.

Conclusions.

The treatment of every foundation offers new and unforeseen problems to which judgment and experience offer the best solution. By reason of its nature, such work does not lend itself to control by rigid specifications and, once the broad plan of approach has been determined, the responsibility for changes necessary to cope with the various emergencies that arise must be placed upon the men in the field. It is seldom that the quality of the work can be permitted to suffer from the interruptions incidental to consultations and studies and the changes are usually in the nature of emergencies. For this reason, it is well to choose inspectors possessing initiative and judgment for directing the work.

The uncertain character of the work makes it impractical to estimate the cost of foundation treatment in advance. It is true that unit costs may be estimated with reasonable accuracy but it is impossible to determine the total quantity of materials that will be required. Having no basis of comparison, it is therefore difficult to judge whether the work is being performed economically. However, by constantly keeping in mind the object of the work and by so regulating the pumping rates and fixing the refusal pressures that the least amount of material, commensurate with the maintenance of the desired standard of quality, is used, appreciable economies may be effected. In practice, this means that a study must be made to determine the characteristics of every hole or group that is to be grouted and that methods must be varied to suit the individual peculiarities of each.

It was realized from the start that the foundation treatment constituted a major problem and that its success depended to a great extent upon the effectiveness with which it would be possible to remove unsound material from the numerous extensive seams and to replace it with grout. The equipment developed for this purpose and for other special uses was found to be highly satisfactory.

CONCRETING [14]

Preliminary cement studies.

Early cement studies were based on an expected time interval of 6 days between successive 5-foot concrete lifts in the dam. Because of this long time interval between lifts, only a small difference in temperature rise between low-heat cement and normal portland could be expected in the mass concrete of the dam. There was some doubt about obtaining groutable joint openings between construction blocks if low-heat cement were used, and there was some concern about the possibility of obtaining a strictly low-heat cement at a reasonable cost. A normal portland cement was considered undesirable because of possible high tricalcium aluminate (C_3A), low fineness, and non-

[14] This discussion of concreting has been taken from TVA Technical Monograph No. 27, Concreting at Norris Dam, by I. L. Tyler, September 1937 (price $1.75).

uniformity in general, and specifications covering two special types of cement, a low-heat cement and a modified portland cement, were prepared.

Both specifications covered such items as theoretical compounds, fineness, and strength, but had no direct requirement for heat of hydration. It was assumed that compound composition would be a sufficiently accurate measure of heat.

The modified type B cement specifications differed only slightly from those for normal portland cement. The essential differences were in a maximum 8 percent tricalcium aluminate (C_3A) requirement, a 1,600–2,200 square centimeter per gram fineness requirement, and 35–55 percent tricalcium silicate (C_3S) limits for type B cement. The type B cement was expected to have a heat of hydration of less than 90 calories per gram in 28 days. This is somewhat lower than that of most normal portland cement, and is due largely to the lower tricalcium aluminate (C_3A).

TABLE 84.—*Cement specifications*

	Type A requirements		Type B requirements		Normal portland (A. S. T. M.) requirements	
	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum
Chemical analysis						
Ignition loss ... percent	----------	3.00	----------	3.00	----------	4.00
Insoluble residue ... do	----------	.85	----------	.85	----------	.85
Sulfuric anhydride (SO_3) ... do	----------	2.00	----------	2.00	----------	2.00
Magnesia (MgO) ... do	----------	5.00	----------	5.00	----------	5.00
Ratio ferric oxide (Fe_2O_3) to aluminum oxide (Al_2O_3)	----------	1.50	----------	1.50	----------	----------
Theoretical compounds						
Tricalcium aluminate (C_3A) ... percent	----------	7	----------	8	----------	----------
Tricalcium silicate (C_3S) ... do	----------	35	35	55	----------	----------
Dicalcium silicate (C_2S) ... do	----------	60	----------	----------	----------	----------
Tetracalcium alumino-ferrite (C_4AF) ... do	----------	20	----------	----------	----------	----------
Physical tests						
Fineness [1]	1,700	2,300	1,600	2,200	----------	22 percent
Soundness	Sound	----------	Sound	----------	Sound	----------
Initial set [2]	1 hour	----------	1 hour	----------	1 hour	----------
Final set [2]	----------	10 hours	----------	10 hours	----------	10 hours
7-day strength	[3] 1,000	----------	[3] 1,500	----------	[4] 275	----------
28-day strength	[3] 2,000	----------	[3] 2,500	----------	[4] 350	----------

[1] Fineness of type A and type B cements measured in terms of specific surface in square centimeters per gram by Wagner turbidimeter. Fineness of normal portland measured by percent retained on 200-mesh screen.
[2] Gillmore needle.
[3] Compressive strengths measured on 2 by 4 test cylinders using a 1–3 mortar of standard Ottawa sand. Later specifications for type B cement permit the use of 2-inch plastic mortar cubes with minimum strength of 750 pounds per square inch at 3 days, 1,500 pounds per square inch at 7 days, and 2,500 pounds per square inch at 28 days.
[4] Tensile strengths measured on mortar briquettes using standard Ottawa sand.

Low-heat type A cement specifications were similar to those for type B except that tricalcium aluminate (C_3A) was lowered to 7 percent, fineness was raised to 1,700–2,300 square centimeters per gram, tricalcium silicate (C_3S) was limited to a maximum of 35 percent, dicalcium silicate (C_2S) was limited to a maximum of 60 percent, and tetracalcium alumino-ferrite (C_4AF) was limited to a maximum of 20 percent. The latter two items were not included in

the type B specifications. Strength requirements were slightly lower for the type A cement.

Detailed specifications covering the two types of cement under consideration are given in table 84. Specifications for normal portland cement as prescribed by the American Society for Testing Materials is also given in the table for comparison.

Consideration was given to the possible use of a blast furnace slag cement as a partial replacement of the portland cement, and extensive tests[15] were made to determine the suitability of such a mixture of cements. Test results were not favorable to the portland-slag cement mixture, and consideration of its use was abandoned for the time. Later it was decided to use a mixture of portland and slag cements in one complete construction block of the dam, thus furnishing a direct comparison by a full-scale test of modified portland cement with a modified portland-slag cement mixture in mass concrete. Details of these tests are given in appendix E.

Final decision to use the modified portland cement instead of the low-heat portland was based on the following considerations:

1. It was known that the type B cement could be manufactured by cement mills in the vicinity with little difficulty, but there was some doubt that the low-heat cement could readily be produced with the raw materials available.

2. Concrete using low-heat cement might cause trouble in cold weather, particularly in thin concrete sections, due to its slower rate of hardening, and the use of two types, one for use in warm weather and the other for use in cold weather, might be found necessary as a result.

3. Concrete containing low-heat cement would require a longer curing period than that using type B cement.

4. There would be a decided advantage in having one cement specification for all Tennessee Valley Authority projects, some of which could not use low-heat cement economically.

Mixes.

During the early stages of construction, mixes determined by early tests were used as far as the aggregates manufactured would permit. The recommended aggregate grading is shown in table 85.

TABLE 85.—*Recommended aggregate grading (by weight)*

Screen size	Recommended grading	Trial grading	Developed grading
	Percent	*Percent*	*Percent*
3-inch to 6-inch	20.0	15.3	17.5
1½-inch to 3-inch	15.2	18.5	16.0
¾-inch to 1½-inch	13.3	13.1	14.0
⅜-inch to ¾-inch	9.5	10.4	11.0
4 mesh to ⅜-inch	6.7	8.3	8.0
8 mesh to 4 mesh	6.6	6.4	8.0
14 mesh to 8 mesh	7.6	9.5	8.0
28 mesh to 14 mesh	6.6	7.3	6.5
48 mesh to 28 mesh	5.5	4.5	3.8
100 mesh to 48 mesh	5.1	2.5	2.7
Minus 100 mesh	3.9	4.5	4.5

[15] Blanks, R. F., U. S. Bureau of Reclamation Technical Memorandum No. 362, Jan. 2, 1934.

In comparing the developed aggregate gradings of concrete mixes with the gradings recommended by early tests, it should be noted that the effective top size of the aggregate used is 5½ inches and not 6 inches as the table indicates. If consideration is given to this difference, the comparisons will be much closer than they appear at first glance.

It was proposed that mass concrete using the recommended grading have a minimum cement content of 1.00 barrel per cubic yard and a maximum water-cement ratio of 0.67 by weight. For concrete in the exposed portion of the dam, the same aggregate grading, a cement content of 1.20 barrels per cubic yard, and a maximum water-cement ratio of 0.60 by weight was recommended. The mass concrete placed in the dam during the first few weeks contained 1.10 barrels of cement per cubic yard and had a water-cement ratio of 0.65 by weight and the face concrete contained 1.25 barrels of cement per cubic yard and had a water-cement ratio of 0.60 by weight.

Trial mixes.—It was necessary to modify the recommended grading at the very start of concreting because of inability to obtain the desired particle size distribution in the fine sand. Experience indicated that particle size distribution in the aggregate had a very decided effect on workability and other qualities of the concrete produced. Aggregate particle sizes between 100-mesh and 28-mesh screens were deficient and could not be produced in sufficient quantity at a reasonable cost. It was found that the effects of this deficiency, poor workability, and excessive "water gain," could be minimized by using what appeared to be an excess of material finer than 100-mesh from which particles finer than 325-mesh had been removed.

From the results of laboratory tests[16] it was pointed out that:

> Although the available data indicate that the presence of fines (minus 100-mesh) up to 20 percent of the sand (minus 4-mesh) has no appeciable effect upon the volume change of concrete due to wetting and drying, there is a general impression that excessive amounts of fines contribute toward the formation of surface crazing and cracking, and laitance layers.

In view of the fact that about 2 to 3 percent more minus 100-mesh material than recommended was to be used, it was deemed advisable to make further tests regarding the relative effect of the amount of fines on volume change. These tests indicated that the additional fines did not appreciably affect the volume change of the concrete due to wetting and drying. Appendix E contains test details.

After about a month's operation during which an attempt was made to use the modified recommended gradation, it was decided to try a new gradation in an attempt to get the most desirable grading possible. This trial gradation is shown in table 85. Laboratory tests and careful observation of concrete behavior in the forms and improvement in control of aggregate manufacture and processing permitted development of a more desirable combined aggregate grading. As aggregate grading, moisture control, and other factors affecting workability and other qualities of concrete were brought under control it was possible to reduce cement content without chang-

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[16] Blanks, R. F., Mass Concrete Mixes for Norris Dam, U. S. Bureau of Reclamation Technical Memorandum No. 368. March 5, 1934.

ing the water-cement ratio. One of the prime factors which made possible the reduction in cement content was the improvement in vibrator efficiency. During the early stages of the job, the vibrators were operated at 67 cycles or about 4,000 impulses per minute. The frequency changers were equipped with interchangeable driving sheaves, making possible frequency changes in steps up to 80 cycles and after about two months' operations the operating frequency was stepped up to 75 cycles or about 4,500 impulses per minute. Another reason for the improved vibrator efficiency was the fact that the operators learned to use the vibrators to better advantage with additional experience gained in concrete placing. In January 1935, about 6 months after concreting started, the vibrator frequency was increased to 80 cycles or about 4,800 impulses per minute. A very decided increase in efficiency was noted at this frequency, especially in stiff concrete.

Changes in trial mixes.—All mixes, particularly those with low cement content, were found to be very sensitive regarding workability and "water gain" caused by the amount of minus 100-mesh material in the aggregate. An excess of this material produced a stiff concrete and a deficiency caused harshness and "water gain." Development of a reclaimer for the fine sand brought the minus 100-mesh material under control and permitted a regrading of the aggregate. The fines reclaimer was first put into operation in December 1934, but it was necessary to experiment with its operation for 2 or 3 weeks before arriving at control methods which would give the desired product. By the middle of January 1935 the reclaimer was operating very satisfactorily, and a trial was made using only 0.90 instead of 0.95 barrel of cement per cubic yard in the mass concrete. The result of this trial was encouraging, but the use of 0.95 barrel was continued until April 1935. The added vibrator efficiency as a result of the change to 80 cycles and the ability to control the minus 100-mesh material made possible a definite change to 0.90 barrel per cubic yard in the mass concrete in April 1935. Significant changes in the face concrete mixes were also made possible by having brought these factors under control. The face concrete was divided into two types; namely, "downstream spillway face concrete," using 1.20 barrels per cubic yard with water-cement ratio of 0.56 by weight, and "regular face concrete," using 1.10 barrels per cubic yard with water-cement ratio of 0.58 by weight. Prior to this change all face concrete contained 1.25 barrels for a very short time at the beginning of construction and 1.20 barrels for the remainder of the time and had a water-cement ratio of 0.60.

The net results of obtaining control of a fair portion of the factors affecting the production and placement of the mass concrete were approximately as follows:

1. Temperature rise in the dam was reduced about 5° F.
2. Strengths were changed little, if any, by the reduction in cement content.
3. Elastic properties were probably changed little, if any.
4. No appreciable effect upon volume change due to wetting and drying as a result of the increase in minus 100-mesh material was produced.
5. A structure of more uniform qaulity was obtained.

6. Durability of face concrete was probably improved by decreasing the water-cement ratio.

7. Cost of cement was reduced by at least $150,000.

Control of mixes.—In order to maintain the control which was so necessary in producing a concrete of uniformly high quality, it was necessary to exercise vigilant inspection of the several processes which combined to produce the concrete in place. There were three control points in the process of production where definite procedures of inspection and testing were maintained to insure proper control. These were the aggregate plant, the mixing plant, and the forms. A fourth measure, which formed a check on the whole control plan, was the laboratory inspection which included the taking of the test specimens.

Variations in aggregate grading were the most difficult to control. During peak production of concrete, both the coarse aggregate and the sand screening plants were greatly overloaded. Wet weather added to screening difficulties, and at times the fine rock screens were badly blinded by damp quarry fines. At such times, the only solution was in using an increased cement content for the concrete. In general, however, gradings of the various sizes remained fairly uniform, and a definite combined aggregate grading could be maintained for extended periods of time, even though the screening was not all that could be desired. Two definite schedules were followed in obtaining samples for screen analysis. One schedule of samples was taken as a basis for controlling the operation of the sand plant and the other was taken as a basis for securing the desired combined aggregate grading for the mix. The following schedule was used in taking samples as a basis for the sand production control:

Number of samples	Type of samples	Time interval	Sample (pounds)
2	Hammer mill feed	Daily	30
2	Hammer mill product	do	15
2	Circulating load	do	15
1	Fine sand (Dorr washer)	2 days	10
1	Fine sand (Link-Belt washer)	do	10

Very careful observations were made during the early operations of the plant as these samples were taken, and from the mechanical analyses and the observations, certain fairly definite facts concerning the operation of the plant were established. After considerable experimenting, it was possible to alter the operation of the sand plant by using the results of the control samples and thus obtain a more desirable product. However, it was not always possible to evaluate in advance all of the factors suggesting certain changes and, throughout the entire job, it was necessary to do some experimenting in order to produce the desired product. However, it was always possible to detect a change in the products and feeds of the sand plant by the samples and also to determine when the proper changes had been made to get the desired product.

The schedule used in obtaining samples for screen analyses to be used as a basis in controlling the combined aggregate grading was as follows:

Number of samples	Type of samples	Time interval	Sample (pounds)
1	Cobbles	Weekly	200–500
2	Fine sand (from reclaiming tunnel)	Daily	
3 [1]	Fine sand (from mixers)	do	
1	Coarse sand (from reclaiming tunnel)	2 days	
1	Fine rock (from reclaiming tunnel)	do	15
1	Medium rock (from reclaiming tunnel)	do	25
1	Coarse rock (from reclaiming tunnel)	do	40

[1] One composite sample each shift composed of part of each moisture determination sample taken during the shift.

Having established a desired grading, the weekly averages of the screen analyses or the averages of the screen analyses best representing the aggregate to be used in the mix were compared to this desired grading, and if very great differences existed which could not be corrected by altering the operation of the aggregate plant, then the proportions of each of the nominal size groups used in the mix were corrected to give the nearest approach possible to the desired grading. Changes were always made on a trial basis, and as additional screen analyses of the aggregate in use became available, more comparisons and additional corrections were made if necessary.

TABLE 86.—*Average screen analyses of sand plant products and concrete aggregates*

[Percent passing]

	Sand plant products week ending Jan. 26, 1935					Concrete aggregates in use as of Jan. 29, 1935					
Screen size	Hammer-mill feed	Hammer-mill product	Circulating load	Coarse sand	Fine sand	Cobbles	Coarse rock	Medium rock	Fine rock	Coarse sand	Fine sand
6-inch						100.0					
5-inch						85.5					
4-inch											
3-inch						12.9	100.0	100.0			
2½-inch	100.0										
2-inch	67.5										
1½-inch	64.3					2.5	6.8	88.8	100.0		
1¼-inch	54.5										
1-inch	43.8	100.0	100.0								
¾-inch	28.4	96.5	97.0			1.2	1.1	8.8	95.2		
½-inch	22.3	93.2	90.5								
⅜-inch	17.7	87.2	77.5			.9	.7	1.5	38.1		
3-mesh	14.1	78.1	56.0	100.0	100.0	.8	.6	1.3	14.1	100.0	100.0
4-mesh	11.4	68.4	37.9	86.0	99.9	.7	.6	1.0	4.2	82.6	99.9
8-mesh	7.1	50.4	14.3	23.4	98.7	.5	.5	.8	2.2	22.6	97.0
14-mesh		37.3	7.1	4.0	69.2	.4	.4	.8	1.6	6.3	66.7
28-mesh		29.1	5.6	3.1	42.4	.3	.4	.7	1.4	4.5	42.0
48-mesh		21.2	4.8	2.9	24.8	.2	.3	.6	1.2	3.8	27.0
100-mesh		17.7	3.9	2.7	11.5	.2	.2	.5	1.1	3.2	14.9
Number samples	5	5	5	2	2	3	6	4	4	10	18

The averages of the sand plant products and feeds for the week ending January 26, 1935, and the averages which best represent the concrete aggregates in use as of January 29, 1935, are given in table 86.

The operation of comparing the actual grading to the desired grading (data given in table 86 may be used as typical case) was as follows:

1. From the proportion of each of the nominal aggregate sizes used in the mix previous to the date under consideration, the amounts of each of the specific sizes actually being used in the mix expressed as percents finer than the indicated screen sizes were computed (table 87).

2. The percentage passing each screen size was plotted on double logarithmic cross-section paper with the horizontal scale divisions representing the clear opening in inches of the square screen sizes and with the vertical scale representing the percent passing these screen sizes. A straight line was then drawn connecting the percent passing the 5-inch screen with the percent passing the 8-mesh screen. This line represented the desired trial grading of the aggregate larger than 8 mesh. The minus 8-mesh desired grading curve was indicated by a second curve. This curve is expressed in percents of the total minus 4-mesh material passing the indicated screen

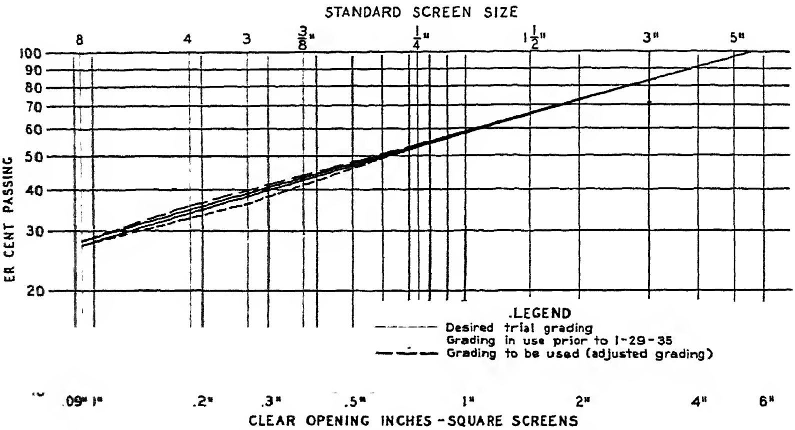

FIGURE 182.—*Combined coarse aggregate grading.*

TABLE 87.—*Mass concrete aggregate grading based on proportions of nominal sizes in use prior to Jan. 29, 1935*

[Mix of Jan. 22, 1935. Percent passing]

Screen size	Fine sand 26 percent	Coarse sand 7 percent	Fine rock 20 percent	Medium rock 13 percent	Coarse rock 14 percent	Cobbles 20 percent	Combined aggregate	Minus 4-mesh [1]	Minus 8-mesh
100-mesh	3.9	0.2	0.2	0.1	--------	--------	4.4	13.4	16.0
48-mesh	7.0	.3	.2	.1	0.1	--------	7.6	23.1	27.6
28-mesh	10.9	.3	.3	.1	.1	0.1	11.8	35.8	42.9
14-mesh	17.3	.4	.3	.1	.1	.1	18.3	55.6	66.5
8-mesh	25.2	1.6	.4	.1	.1	.1	27.5	83.6	100.0
4-mesh	26.0	5.8	.8	.1	.1	.1	32.9	100.0	--------
3-mesh	26.0	7.0	2.8	.2	.1	.2	36.3	--------	--------
⅜-inch	--------	33.0	7.6	.2	.1	.2	41.1	--------	--------
¾-inch	--------	33.0	19.1	1.1	.2	.2	53.6	--------	--------
1½-inch	--------	--------	53.0	11.5	.9	.5	65.9	--------	--------
3-inch	--------	--------	--------	66.0	14.0	2.6	82.6	--------	--------
5-inch	--------	--------	--------	--------	80.0	17.1	97.1	--------	--------
6-inch	--------	--------	--------	--------	--------	100.0	100.0	--------	--------

[1] F. M. = 2.88.

size. The curve representing the grading of the material in the combined aggregate was also shown. The minus 4-mesh desired grading was developed on the job after considerable experimenting in the field. It represents the best grading which could be developed with the material which this plant produced. Laboratory tests indicated that the grading recommended by the early tests, had it been obtainable, would have been the more desirable of the two; but, as already pointed out, the deficiency in the particle sizes between 28-mesh and 100-mesh necessitated the change to the developed grading.

3. If a variation from the desired grading occurred, a new set of proportions of the nominal sizes (table 88) was selected by trial which, when combined, gave the closest possible approach to the desired grading.

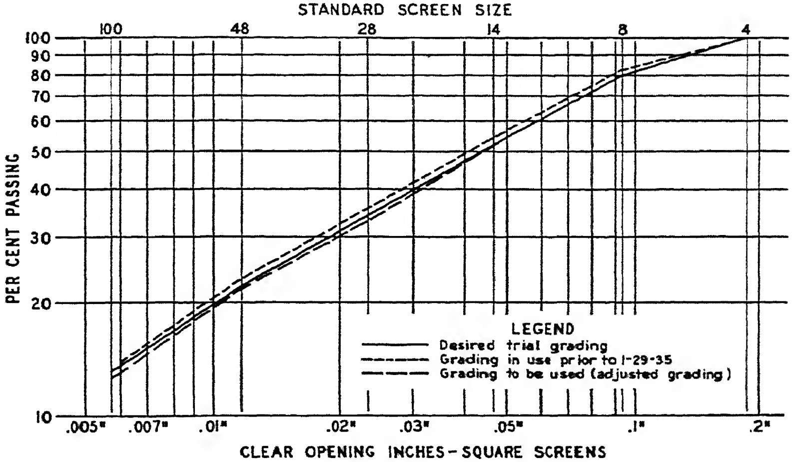

FIGURE 183.—*Combined fine aggregate grading.*

TABLE 88.—*Mass concrete aggregate grading based on proportions of nominal sizes to correct to desired grading*

[Percent passing]

Screen size	Fine sand 26 percent	Coarse sand 10 percent	Fine rock 17 percent	Medium rock 14 percent	Coarse rock 13 percent	Cobbles 20 percent	Combined aggregate	Minus 4-mesh[1]	Minus 8-mesh
100-mesh	3.9	0.3	0.2	0.1	-------	-------	4.5	12.7	16.0
48-mesh	7.0	.4	.2	.1	-------	-------	7.7	21.8	27.3
28-mesh	10.9	.5	.2	.1	0.1	0.1	11.9	33.7	42.2
14-mesh	17.3	.6	.3	.1	.1	.1	18.5	52.4	65.6
8-mesh	25.2	2.3	.4	.1	.1	.1	28.2	80.0	100.0
4-mesh	26.0	8.3	.7	.1	.1	.1	35.3	100.0	-------
3-mesh	26.0	10.0	2.4	.2	.1	.2	38.9	-------	-------
⅜-inch	-------	36.0	6.5	.2	.1	.2	43.0	-------	-------
¾-inch	-------	36.0	16.2	1.2	.1	.2	53.7	-------	-------
1½-inch	-------	-------	53.0	12.4	.9	.5	66.8	-------	-------
3-inch	-------	-------	-------	67.0	13.0	2.6	82.6	-------	-------
5-inch	-------	-------	-------	-------	80.0	17.1	97.1	-------	-------
6-inch	-------	-------	-------	-------	-------	100.0	100.0	-------	-------

[1] F. M. = 2.99.

At the time represented by this example, the "fines reclaimer" was still in the experimental stage; also, the fine rock and coarse sand contained excessive amounts of minus 3/8-inch and minus 4-mesh material respectively, which with more efficient screening later were eliminated to a large extent. As a result, it was necessary to reduce the coarse sand in order to avoid an excess of minus 8-mesh material in the combined aggregate; then, in order to have sufficient minus 3/8-inch plus 8-mesh material, this coarse sand was replaced by an equal amount of fine rock. Also, in order to satisfy the minus 100-mesh requirement, it was necessary to use from 1 to 2 percent more fine sand than was used later when the fines reclaimer was in better operation. When the fines reclaimer was finally in proper operation and the screening of the fine rock and coarse sand was brought under control, it was possible to increase the coarse sand about 3 to 4 percent, decrease the fine rock an equal amount, decrease the fine sand about 1 to 2 percent and increase the cobbles a like amount. These changes coincided with the change in the cement content of mass concrete from 0.95 barrel of cement per cubic yard to 0.90 barrel per cubic yard.

In addition to the aggregate production control samples and the aggregate grading control samples, one monthly composite sample made up from the three daily composite samples of fine sand from the mixing plant was analyzed chemically as a check on the uniformity of the quarry. The final decision to use sand manufactured from dolomite in the exposed portions of the dam was based on laboratory tests which indicated that a practically pure, uniform, and high quality dolomite could be expected from the entire quarry. These samples substantiated this expectation.

Control of the moisture at the mixing plant depended on accurate and rapid moisture determination on the aggregates. It depended also on uniformity of moisture in the various sizes. Fine sand presented the most difficult problem since it was stock piled very wet. By observing a definite routine of stock piling and by always drawing from that portion of the stock pile which had been well drained, sand with a reasonably uniform moisture content was delivered to the mixing plant bins. Moisture determinations were made on samples taken from the batcher by weighing a representative sample before and after drying in an electric oven. An indicating voltmeter on the dispatcher's board, connected to two plate elements in the fine sand batcher, measured the electrical resistance of the sand and thus indicated sudden moisture content changes. By continuous comparison between voltmeter readings and moisture determinations, sudden moisture changes could be detected in the fine sand, and the mixing water corrected before the concrete batch to be affected was put into the mixer. Moisture determinations on the five larger aggregate sizes were made entirely by oven drying. Variations in the moisture in the larger sizes were small, except in a few isolated cases.

In addition to all of the control measures up to and including the mixing process, constant observation of the mixed concrete as it appeared in the transfer trains and as it appeared when being placed in the forms was maintained to detect, if possible, changes in the moisture or aggregate. It was possible on many occasions to detect changes in both moisture and aggregate grading which had not been

disclosed by the routine control measures. Visual inspection played a definite part in the control plan.

Uniformity of cement played an important part in the control of concrete quality. At no time during the construction of the dam was it possible to detect changes in workability of the concrete traceable to variations in physical properties of the three different commercial brands of type B cement used. Importance of uniformly high fineness was very obvious in controlling workability and "water gain" in the concrete. The average fineness of all of the type B cement used in the structure was 1,820 square centimeters per gram (by Wagner turbidimeter), and the uniformity of grinding is indicated by the fact that the fineness of most of the cement furnished fell within the range between 1,750 and 1,900 square centimeters per gram. Had large variations in cement fineness occurred, the value of closely controlled sand grading would have been appreciably decreased. Close control of chemical properties of the cement aided in providing concrete of uniform quality throughout the structure. Heat of hydration, although not directly limited by the specifications, was considerably lower and much more uniform than could reasonably be expected of normal portland cements. Early strengths of concrete varied somewhat with the permissible variations in composition and fineness of the cement, but tests at 28 days and longer indicated reasonably close comparisons.

Mixing water was taken directly from the Clinch River and was used without treating. During high river stages, the water carried an appreciable amount of silt, and some concern was felt about possible presence of organic matter in detrimental amounts. Color tests on concentrations of the silt indicated only a small amount of organic matter, however, and no appreciable effect on concrete strength could be found when the dirty water was used. Tests were made during the most unfavorable conditions, and as the extremely high river stages were rare, the effect of undesirable material introduced into the concrete through the mixing water was undoubtedly negligible.

Details of mixes.—Five general types of concrete mixes were used in the dam and powerhouse. Modification of these and special mixes were also used where conditions required concrete having qualities not obtainable in any of the five standard mixes. Water-cement ratio determined the cement requirement of all mixes except those used in exposed portions of the dam, where both minimum cement content and maximum water ratio were specified. In order to avoid confusion and to prevent mistakes and delays in mixing and placing, concrete was confined to one of the five standard mixes as far as possible.

Mass concrete, including all concrete in the interior of the dam which was not reinforced, made up the bulk of all concrete yardage placed. This concrete had a maximum water-cement ratio of 0.67 by weight, and though no minimum cement content was specified, it was originally assumed that 1.00 barrel per cubic yard would be as low as the maximum water ratio would permit for proper placement. The maximum nominal size of aggregate in this concrete was 6 inches. This "protected interior" concrete offered the greatest possibility for saving in cement and was investigated thoroughly for means of lowering its cement content. As previously noted, the

cement content was successively reduced from 1.10 to 0.90 barrels per cubic yard by bringing under better control the factors affecting its manufacture and placement. It is very probable that cement content could have been reduced still more without change in the water-cement ratio had it been possible further to correct aggregate grading and maintain more accurate control of all mixing and placing operations.

For a thickness of from 6 feet to 12 feet from exposed faces, concrete having an appreciably lower water-cement ratio than that of the interior mass concrete was used. Two mixes were used, one for the upstream faces and the nonoverflow sections, and the other for the spillway faces of the dam. Both mixes contained 6-inch maximum size aggregate. The division of "face concrete" into two classes was made after it became apparent that the specified maximum

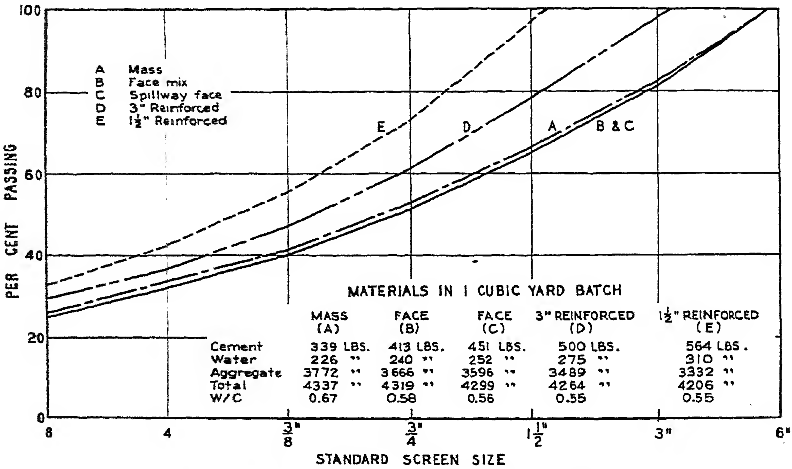

FIGURE 184.—*Concrete grading and mix data.*

water-cement ratio could be appreciably decreased without changing the specified minimum cement content of 1.20 barrels per cubic yard. It was also possible to maintain the water-cement ratio of 0.58, which was below the specified maximum of 0.60, and at the same time reduce the cement content. As a result, the two mixes were adopted as follows: For "spillway face concrete", the cement content was maintained at 1.20 barrels per cubic yard and the water-cement ratio was reduced to 0.56 by weight; and for other "face concrete" the water-cement ratio was maintained at 0.58 by weight and the cement content was reduced to 1.10 barrels per cubic yard. The reduction in cement content of concrete in all but spillway faces represented a considerable saving in cement cost, and the reduction in water-cement ratio of concrete in the spillway faces appreciably increased the quality with no increase in cost.

Reinforced concrete sections of moderate thickness having reinforcing steel 10 inches or more on centers were made of concrete

having 1.33 barrels per cubic yard, a water-cement ratio of 0.55 by weight and 3-inch maximum size aggregate. This class of concrete was used in the reinforced training walls, large reinforced foundations, and spillway faces where the necessity for screeded surfaces or reinforcing made the use of the "spillway face mix" with 6-inch aggregate impossible. Reinforced sections, such as powerhouse walls and heavily reinforced foundations, were made of concrete having 1.50 barrels of cement per cubic yard, a water-cement ratio of 0.55, and a maximum size aggregate of 1½ inches. This mix made up the bulk of all reinforced concrete placed. A reinforced mix using 1.70 barrels of cement per cubic yard, water-cement ratio of 0.55 by weight, and ¾-inch maximum size aggregate was used in the powerhouse floors and walls where reinforcing steel and conduit were so closely spaced that it was impractical to use 1½-inch maximum size aggregate. This mix was also used in the spillway bridge parapet walls. A combined total of all of this type concrete represented less than 200 cubic yards. A special reinforced mix using 1.70 barrels of cement per cubic yard, water-cement ratio of 0.50 by weight, and 1½-inch maximum size aggregate was used for the penstock intake structures and transition section and sluice tube intakes.

Mortar for use on day's work joints contained approximately 2.80 barrels of cement per cubic yard and had a water-cement ratio of 0.53 by weight. The mix was 1 to 2.1 using both fine and coarse sand to ⅜-inch size.

A wide range in quality existed between the concrete used for the penstock and other similar structures and that used in temporary gravity section cofferdams. The mix for cofferdams contained 0.75 barrel of cement per cubic yard and had a water-cement ratio of 0.90 by weight and 6-inch maximum size aggregate. Such concrete had a high "water gain" and was harsh and difficult to place without excess mixing water, but was found to be sound and tough at the times of demolishing the structure in which it was used. Characteristics of the two extreme mixes are as follows:

Water-cement ratio by weight	Cement content	Maximum size aggregate (inches)	Crushing strength—Pounds per square inch			
			7 days	14 days	28 days	90 days
0.90 [1]	0.75	6	948	1,060	1,648	2,303
.50	1.70	1½	3,321	4,464	5,699	6,812

[1] Water-cement ratio not closely controlled.

Strength of field-mixed concrete.—Concrete strengths using dolomite aggregates and type B cement have been somewhat higher than water-cement ratios would indicate. It was known that the type B cement had slightly higher than normal strength qualities, but strengths obtained were higher than could be accounted for by this factor alone; therefore, physical or chemical properties of the dolomite must have contributed in appreciable measure. Compressive strength data on 6- by 12-inch test cylinders of field-mixed concrete are shown in table 89. The test data include strengths of concrete cylinders containing any one, or all three of the brands of cement

TABLE 89.—*Average strength of field mixed concrete Dec. 1, 1936*

MASS (6-inch aggregate—type B cement)

Barrels of cement per cubic yard	1.10		1.00		0.97		0.95		0.90	
Water-cement ratio	.65		.67		.67		.67		.67	
Age	Number of cylinders	Average strength	Number of cylinders	Average strength	Number of cylinders	Average strength	Number of cylinders	Average strength	Number of cylinders	Average strength
7 days	108	2,209	74	2,529	10	2,163	354	2,689	228	2,695
14 days	41	3,169	23	3,203	2	3,608	189	3,523	178	3,485
28 days	104	4,159	68	4,047	6	4,643	306	4,483	227	4,404
90 days	108	5,597	60	5,488	4	5,350	347	5,810	256	5,736
6 months	39	6,621	23	6,060	3	6,957	169	6,397	113	6,396
1 year	78	6,690	27	6,513	6	7,027	173	6,869	110	6,876
2 year	---	---	27	6,851	---	---	---	---	---	---

FACE (6-inch aggregate—type B cement)

Barrels of cement per cubic yard	1.25		1.20		1.20		1.20		1.10	
Water-cement ratio	.60		.60		.57		.56		.58	
Age	Number of cylinders	Average strength	Number of cylinders	Average strength	Number of cylinders	Average strength	Number of cylinders	Average strength	Number of cylinders	Average strength
7 days	10	2,524	46	2,833	98	3,617	35	3,448	60	3,522
14 days	6	3,743	21	3,810	51	4,646	33	4,483	57	4,426
28 days	14	4,858	48	4,730	82	5,500	48	5,460	99	5,360
90 days	12	6,488	66	6,328	104	7,332	54	6,862	96	6,856
6 months	6	7,690	21	7,057	41	7,768	18	7,296	15	7,313
1 year	7	7,597	38	7,697	48	8,609	15	7,963	15	8,099

REINFORCED (type B cement)

Barrels of cement per cubic yard	1.50		1.50		1.70		1.33		1.48	
Water-cement ratio	.57		.55		.50		.55		.60	
Maximum size of aggregate	1½ inches		1½ inches		1½ inches		3 inches		1½ inches	
Age	Number of cylinders	Average strength	Number of cylinders	Average strength	Number of cylinders	Average strength	Number of cylinders	Average strength	Number of cylinders	Average strength
7 days	14	2,762	37	3,319	11	3,320	21	3,431	6	2,453
14 days	7	3,639	27	4,214	8	4,464	15	4,608	6	3,230
28 days	18	4,659	40	5,007	13	5,699	26	5,560	12	4,085
90 days	16	5,494	36	6,498	6	6,812	20	7,087	12	5,451
6 months	5	6,620	8	7,325	4	8,025	3	8,763	---	---
1 year	9	6,951	8	7,915	3	8,243	2	8,755	---	---

NOTE.—All concrete having aggregate larger than 1½ inches was wet screened through 1½-inch square opening wire screen; rate of loading, 17 pounds per square inch per second; all cylinders are 6 by 12 inches; all tests complete through 1 year.

used. Comparison of the test results furnish striking evidence of what may be accomplished by controlling operations in the manufacture and placing of concrete.

As an additional check on quality of concrete actually placed in the dam, 17- by 34-inch cores were cut from several locations in the dam and tested for elastic modulus, crushing strength, and other properties. The strength of concrete as placed in the dam compared very favorably with that predicted from laboratory investigations. Table 90 gives essential data on properties of the specimens tested.

The unit weight of the concrete was high because of the heavy aggregate used. The average specific gravity of the aggregate was 2.82. Calculated weights checked closely with measured weights of cores taken from the dam. Mass concrete in place weighed 160 pounds per cubic foot, and concrete containing 1½-inch maximum aggregate, used in reinforced sections, weighed 156 pounds per cubic foot.

Placing.

Concrete in the dam was placed in 5-foot lifts in construction blocks generally 56 feet in width and up to 200 feet in length, depending on the elevation of the lift. Surfaces of each lift were sloped 5 percent downward toward the upstream face of the dam.

A 3-day interval between consecutive lifts was provided to permit dissipation of a part of the heat liberated by the cement during hardening. No restrictions were placed on differences in elevation between adjacent blocks although a three-lift, 15-foot difference in elevation was maintained for convenience when conditions permitted. A concrete lift once started was carried to completion except in very rare cases of mechanical or electrical failure of equipment or impossible working conditions. Exceptions to the time limit and slope requirement were made in a few locations. For instance, in the closure blocks (40 and 43) several 5-foot level lifts were placed at approximately 48-hour intervals. In some sidehill locations, minimum placing intervals of 7 days were required.

Concrete was deposited only on thoroughly cleaned surfaces of foundation rock or previously placed concrete. Surfaces were sprinkled, and the excess water was blown off with compressed air just before concrete placing was started. Except in unusual circumstances, concrete placing proceeded from the upstream end toward the downstream end of the construction blocks, since by placing against the slope of the preceding lift, working faces could be comparatively short and still have reasonably flat slopes. Steep placing slopes were avoided. An additional advantage of this procedure was that completed surfaces of the lift could be cleaned with air and water without disturbing placing operations.

Concrete placing with one cableway operating in one construction block, although varied to suit special conditions, generally followed a set procedure. The first bucket sent to the form contained mortar which was deposited approximately 15 feet from the upstream face of the dam. The mortar was spread over an area extending about 25 feet from the upstream form and thoroughly brushed into the old concrete surface with wire brooms. Mortar was followed by

Table 90.—*Test results on 17- by 34-inch concrete drill cores from mass concrete in Norris Dam—age 180 days*

[From U. S. Bureau of Reclamation Technical Memorandum 481, table 3]

	Core number								
	35A	35B	Average	44A	44B	Average	45A	45B	Average
Coordinates:									
West	16+85	16+93		21+07.0	21+87.0		22+40	22+40	
South	21.75	27.50		121.3	122.8		72.0	82.0	
Elevation at top	840	840		835	835		894.6	895.1	
Temperature at time of removal	43.5° F.[a]	44.5° F.[b]		43° F.[a]	44° F.[b]		43° F.[a]	44° F.[b]	
Cement—									
Barrels per cubic yard	1.10	1.10		1.00	1.00		0.95	0.95	
Water-cement ratio by weight	0.65	0.65		0.67	0.67		0.67	0.67	
Date placed, 1934	Sept. 12	Sept. 12		Oct. 13	Oct. 13		Nov. 12	Nov. 12	
Date removal, 1935	Feb. 2	Feb. 2		Jan. 29	Feb. 5		Jan. 30	Jan. 30	
Date tested, 1935	Mar. 12	Mar. 12		Apr. 12	Apr. 12		May 13	May 15	
Compressive strength, pounds per square inch	5,150	5,120	5,135	4,560	4,260	4,410	5,220	5,000	5,110.
Modulus of elasticity, 800 pounds per square inch.	4.2 x 10^6	4.7 x 10^6	4.45 x 10^6	4.8 x 10^6	4.9 x 10^6	4.85 x 10^6	5.5 x 10^6	6.2 x 10^6	5.85 x 10^6.
Poisson's ratio, 800 pounds per square inch	0.17	0.17	0.170	0.18	0.18	0.180	0.26	0.27	0.265.
Weight per cubic foot	157.6	160.5	159.1	159.3	157.3	158.3	159.2	158.8	159.0.

[a] Top.
[b] Bottom.

Note.—The estimated strength and modulus of an 18- by 36-inch laboratory-fabricated concrete cylinder 180 days old, with 6-inch maximum Norris Dam aggregate, 0.67 water-cement ratio, 1.02 barrels blended modified cement per cubic yard, and 1:3.33:6.17 mix are 4,100 pounds per square inch and 5,100,000 pounds per square inch, respectively. The 28-day results of 18- by 36-inch cylinders containing the above ingredients plus the 28- and 90-day results of 18- by 36-inch cylinders for both Arizona and Grand Coulee aggregate containing 1.0 barrel blended modified cement per cubic yard, 6-inch maximum aggregate, plus the 28-, 90-, and 365-day results of 3 Boulder Dam mass mixes were used in the estimation. An estimation using 28-, 90-, and 365-day results of 6- by 12-inch cylinders with curves and data from paper by R. F. Blanks and C. C. McNamara, "Mass Concrete in Large Cylinders"—Journal A. C. I., February 1935—gave slightly higher values.

mass concrete which was deposited 15 or 20 feet from the upstream end of the block to form a ridge about 2 feet high extending the width of the block. The upstream toe of the ridge was 5 feet or more from the upstream form and the downstream toe usually about 25 feet from the upstream form, or far enough to cover completely the mortar previously spread. Face mix concrete was then deposited between the ridge of mass concrete and the upstream form to a depth of about 2 feet, after which mass concrete placing was resumed until the ridge had been sufficiently raised to permit continuing with the face concrete. By alternating mixes in this manner, the average thickness of face concrete could be held to less than 10 feet, and a thoroughly bonded interlocking joint provided between the mass concrete and the more durable concrete at the exposed face. By the time the upstream form had been filled, a fresh concrete surface, 25 or 30 feet long, sloping about one in eight downward away from the upstream face, had been developed. This slope was maintained by depositing concrete first at the toe of the slope, then working back up the slope in such a manner that concrete was always being deposited on nearly horizontal surfaces. Mortar was deposited and brushed into the old concrete surface as needed to maintain an adequate placing area. Proper size of the placing area depended on concrete temperatures and weather conditions, long slopes being desirable in cool weather and shorter ones in warm weather. When concrete placing had progressed to a location such that the toe of the placing slope came within about 6 feet of the downstream face form, face mix concrete was deposited as near to the form as possible. Alternation of mixes similar to the procedure employed at the upstream face provided a layer of face concrete at the sloping downstream face of the dam averaging about 10 feet in thickness. Two-cableway operation sometimes disrupted this generally accepted best method of concrete placing, particularly when operations were confined to thin sections near the top of the dam. However, careful scheduling of concrete placing and skillful cableway operation minimized the difficulties.

Vibrators.—Under normal placing conditions, four VS4 internal vibrators were required to compact thoroughly all concrete delivered by one cableway (80 to 90 cubic yards per hour). Immediately after deposit, each pile of concrete was attacked by all four vibrators which were moved from one location to another as required until the pile was completely vibrated. Usually a vibrator was left vibrating in one position for 10 to 20 seconds before being moved to a new position 18 inches or 2 feet away. After vibration, a pile of concrete which had originally been 3 feet or more in height was reduced to 18 inches or less and so blended with adjacent concrete that the individual pile could not be distinguished. With fresh concrete underneath, vibrators were inserted deep enough to penetrate the concrete below, insuring a complete blending of the new concrete with that previously placed. The first concrete placed on the old surface was vibrated close to the joint with vibrators nearly horizontal to aid in obtaining a bond between the new and the old concrete.

For thin walls and heavily reinforced sections, Mall electric vibrators with vibrating elements attached to flexible shafts and Viber air-driven vibrators also with vibrating elements attached to flexible

shafts were used. The Mall vibrators, operated by the 80-cycle power provided for the Electric Tamper and Equipment vibrators, were very powerful machines, but were very awkward to use and required excessive repairs. The Viber equipment was easily handled and, though not so powerful as the Mall vibrators, had sufficient power for placing reinforced concrete in thin sections. Repairs to shafts were frequent, and a wet or dirty air supply immediately put these machines out of commission entirely or greatly reduced their effectiveness. Practically all concrete in thin sections of the powerhouse and dam were placed with the aid of either Mall or Viber vibrating equipment, and though neither was all that could be hoped for in operating efficiency, the high maintenance cost was small compared with the cost of placing concrete of equal quality by hand tamping methods.

Clean-up.—Foundation rock and concrete surfaces to be covered by concrete received careful treatment to insure proper conditions at the time of concrete placing. Rock surfaces, after approval for struct-

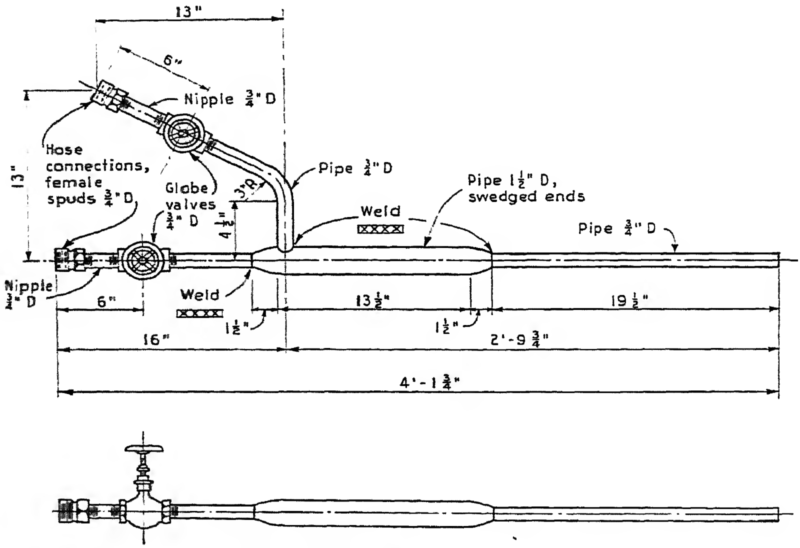

FIGURE 185.—*Nozzle used to clean concrete surfaces.*

ural soundness, were thoroughly washed to remove all clay and foreign material and in some cases chipped and wire brushed to insure a surface to which the concrete would bond. Procedure varied greatly for the different conditions encountered.

Concrete surfaces received a routine treatment. The clean-up operation began between 6 and 18 hours after the top surface of a concrete lift had been completed, actual time depending on concrete placing temperature, air temperature, and the concrete mix being placed. With the concrete surface at the proper stage of hardening,

a jet of air and water forced together through a common nozzle was directed against the surface to be treated. The high velocity jet removed the thin surface film leaving clean rock and mortar surfaces exposed. By proper manipulation of the jet, rock particles were left undisturbed embedded in the mortar. The process required constant attention on the part of operators to insure a proper surface, since cutting too soon removed too much mortar, loosened the larger rock particles, and left a poor surface to receive the fresh concrete. If delayed too long, the cutting action of the air and water jet was ineffective, and other means of preparing the surface were necessary.

In some locations, use of the air and water jet was not feasible, and wire brooming of surfaces was necessary. Such wire brooming also had to be done at just the proper stage of surface hardening in order to obtain a satisfactory surface. It is believed that the air and water clean-up produced the most satisfactory surface but that the wire brushing, if properly done, produced an acceptable surface.

Concrete surfaces left exposed for extended periods required special treatment. Such surfaces usually became stained and coated with deposits which could not be removed by washing or wire brooming. Jackhammers fitted with large bits were used to chip such surfaces, and while the amount of time required to cover completely large areas was excessive, the results obtained were satisfactory.

All concrete surfaces required at least one more washing, usually with air and water, after the initial cleaning to remove waste left by form building for the following lift. In the cutting and washing of concrete surfaces, particular attention was given to areas around the vertical copper water stops near the upstream face of the dam, since in these locations possible percolation paths were the shortest. The upstream 20 feet of each construction block received only slightly less attention.

Air and water were provided at convenient locations on the dam for cleaning, sprinkling, and other uses. For convenience, 2-inch pipes, cast in the concrete, were used as supply lines, 5-foot risers being added for each 5-foot lift of concrete.

Curing.—Curing of concrete consisted of the usual manual sprinkling with water hose and a few installations of fixed sprinkling systems. When feasible, horizontal surfaces were covered with damp fine sand, since this decreased the amount of sprinkling required and assured a damp surface. Sprinkling of completed walls and other permanently exposed surfaces by means of fixed sprinkling systems, using the available river water containing an appreciable amount of silt, at times caused undesirable staining which could be removed only with great difficulty; consequently, hand sprinkling, less desirable from the curing standpoint, came into general use. The curing period was 28 days.

Cold weather added complications to curing of concrete. Two small vertical boilers supplied steam for cold weather concrete placing in the powerhouse and in some sections of the penstock intake structures. Concrete in the dam was protected from freezing by tarpaulins placed over the completed lifts aided by salamanders. However, the use of salamanders was very unsatisfactory. In spite of all precautions, concrete directly under the salamanders would occasionally dry out with resulting damage to the concrete.

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Personnel.—Concrete placing, clean-up, and curing were under the direct supervision of a general concrete foreman assisted by two shift foremen, one on each of the afternoon and night shifts. Cableway, mixing plant, and transfer system operations were under the general concrete foreman's general supervision, although workmen in these plants reported directly to the rigging and mechanical departments. Four 6-hour shifts for workmen were maintained during the greater part of the job. Foremen worked on 8-hour shifts. Operations were ordinarily on a 6-day week basis.

During the major part of concrete placing operations when both cableways were being used, a maximum of 90 cubic yards of mass concrete per hour was placed by each cableway. A crew normally consisted of a foreman and eight men. One man operated the pneumatic bucket dumping device as loads of concrete were received and helped operate a vibrator the remainder of the time. Extra men were added to the crew when difficult placing conditions were encountered. Crews were increased to 10 or 12 men when only one cableway was used, and the rate of concrete placing was 120 cubic yards per hour or more. Concrete placing crews filling special forms in the dam or in the powerhouse varied in size as placing rates and placing conditions demanded. In general, more men were required to place special pours than for mass concrete placing. One carpenter and a helper were assigned to each mass concrete placing crew to do such jobs as check forms, set form anchors, and do the flat vibrator work required at the top of each lift. Two electricians divided their time between the vibrators, the mixing plant telephones, and the cableway telephones and other signalling apparatus.

The clean-up work was supervised by one foreman on each of the three 8-hour shifts. Two to six men were ordinarily required in each form. During the early stages of construction, three additional foremen, one on each shift, were needed for preparation of foundation surfaces. Men for this work were transferred temporarily from other labor crews as working conditions warranted.

Curing of concrete required as many as six laborers per shift to do the necessary sprinkling. No regular organization was created for this work, men being drawn from other crews as needed.

Forms.—Wooden panel forms were used for plane concrete surfaces of the dam. The forms were built in the carpenter shop complete and ready for use. Panels for exposed faces of the dam were 14 feet long and for contraction joint faces were 20 feet long. Shorter filler sections were used as required. Contraction joint-forms were built as parallelograms to provide the 5-percent slope required of concrete lifts, the joints remaining vertical. Keyway forms of comparatively light construction were nailed and bolted to the contraction joint panels.

Two sets of panels were required for each concrete surface, the form to be placed being supported by the form which had been filled with concrete. Studding of the top form was held by the top wale of the lower form on one side, but support on the other side was lacking; consequently, the studs could not be wedged tightly in place before concrete placing was started. There was chance of small horizontal movement of the upper form away from the fresh concrete unless wedges between the wale and studs were driven and kept

tight as depth of lift increased. Wearing of studding with repeated use added to the difficulty of maintaining accurate alignment. Short pipe braces between the old concrete and lower part of the forms were sometimes used to permit wedging before concrete placing was started.

Tops of forms were aligned and held in position by tie rods and pipe braces. Tie rods were ¾-inch in diameter, hooked on one end and threaded on the other. Three-fourths-inch hooked rods embedded in fresh concrete at the top of each lift provided anchor-

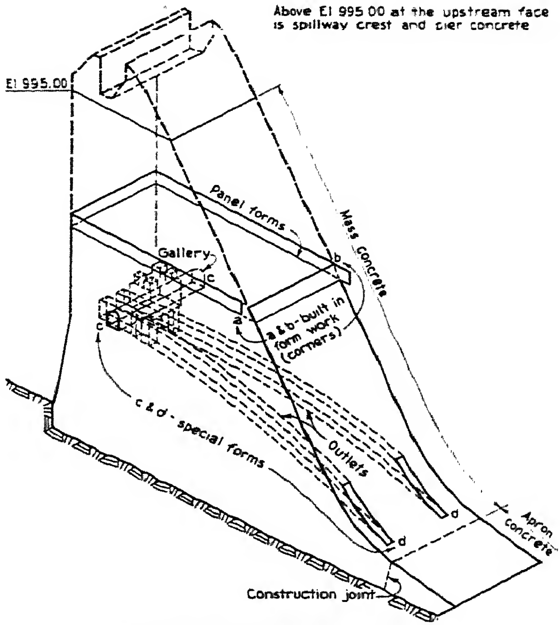


FIGURE 186.—*Form nomenclature.*

age for the tie rods. Sleeve bolts through the top wale, tapered and greased to permit removal from the hardened concrete, provided adjustment in length of tie rods. Auxiliary ⅝-inch anchors were provided to hold forms during dismantling after the large sleeve bolts had been removed. Smaller sleeve bolts were used for these anchors. Braces made of 1½-inch salvage pipe wedged against removable blocks nailed to the forms were placed beside each tie rod. Pipe braces were removed for re-use after approximately 3½ feet of concrete had been placed against the forms. Forms were placed by special equipment adapted to the purpose. Three-wheel rubber-tired Krane Kars with short boom and hoisting mechanism lifted panels from the old to the new positions. A special jack was first attached to the panel, it then pulled the panel horizontally

away from the concrete surface and held it for lifting to its new position. Krane Kars and lifting mechanism were moved from one block to another by means of the cableway.

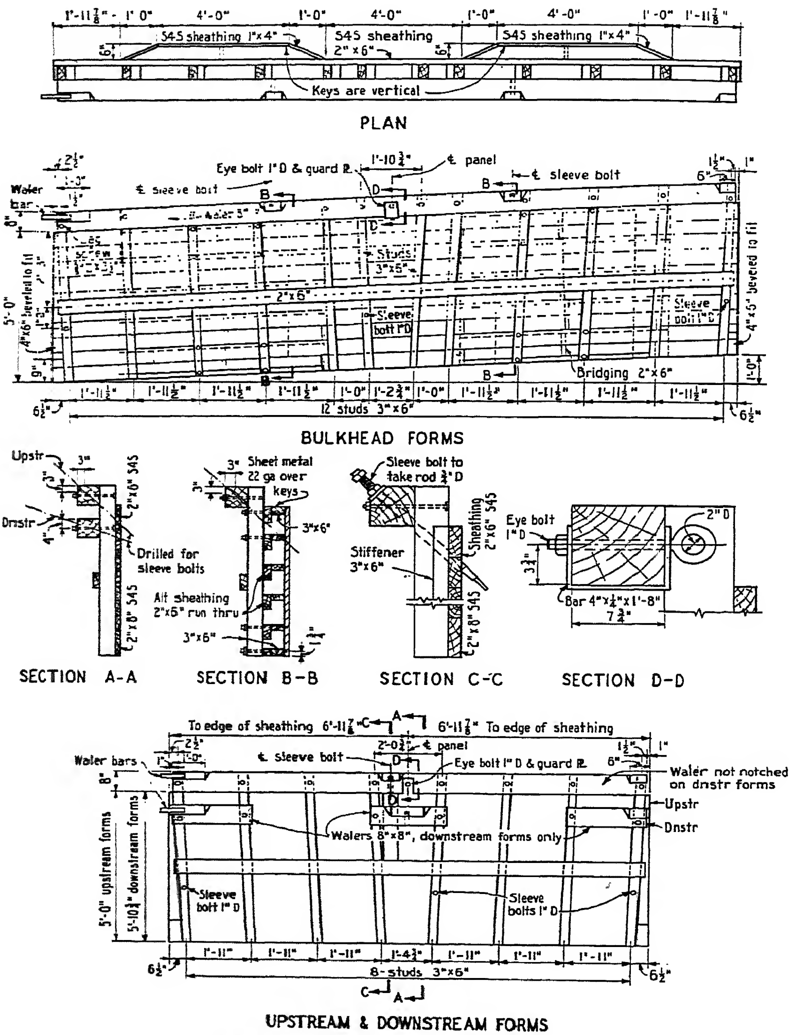


FIGURE 187.—*Typical panel forms.*

The form panels were very satisfactory in general. A few of their shortcomings have been mentioned above. There are a few others, most of which could be remedied. During the first use,

leakage of mortar between lagging boards was appreciable, but in subsequent uses the leakage was small since the joints were filled by mortar. In some cases panels were used 20 times or more; however, the last uses showed unsatisfactory concrete surfaces because of poor condition of the forms.

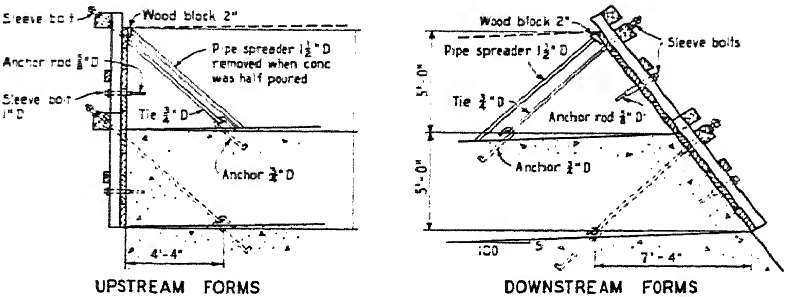


FIGURE 188.—*Form bracing.*

Special forms for such structures as trashracks, roadway, spillway bridge, and draft tubes, will be described in the sections which deal with these structures.

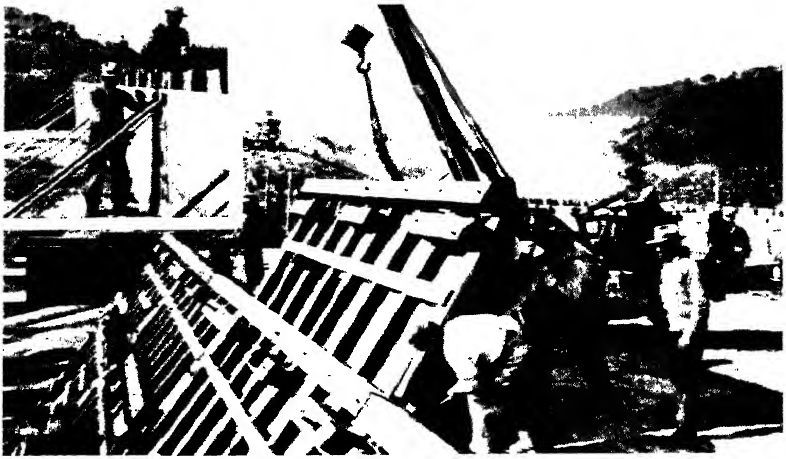


FIGURE 189.—*Placing panel forms.*

Closure sections.—Construction blocks 40 and 43 were used for final closure of the dam. For river diversion concrete in these two blocks was placed in horizontal layers up to elevation 875 instead of the usual 5 percent slope. Lifts were approximately 5 feet thick with 2½-foot differences in elevations between surfaces of lifts in the two blocks during the closure process.

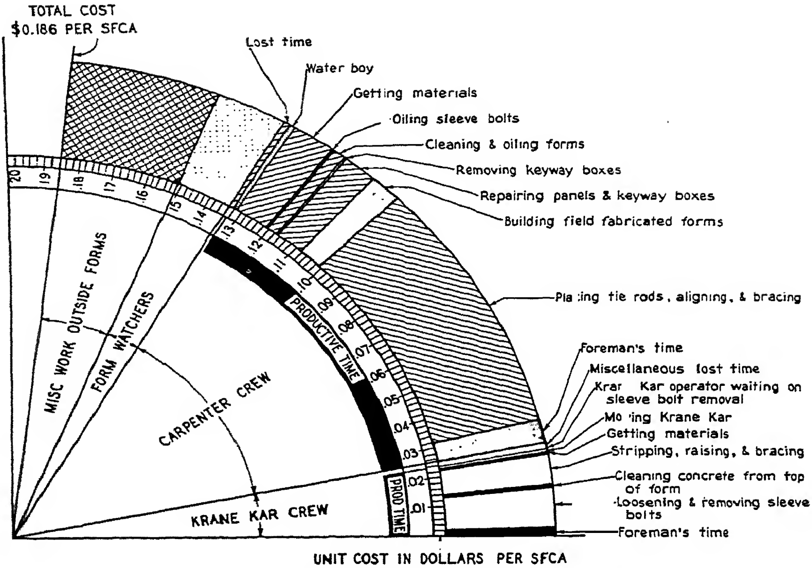


FIGURE 190.—*Field labor costs of panel forms (based on special time studies).*

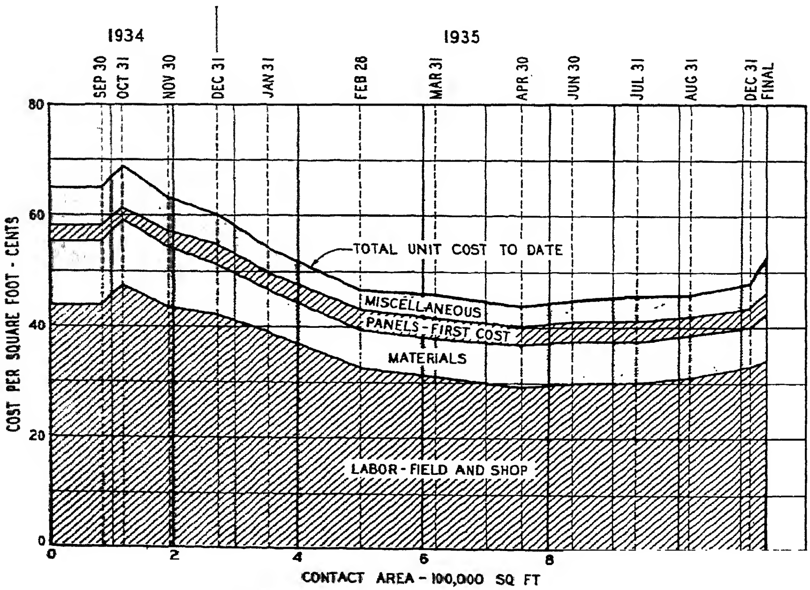


FIGURE 191.—*Formwork costs.*

The closure method used was very successful. After the first lift or two, the time required for completion of one 4-foot 10-inch lift in both blocks was only about 4 days. Little difficulty with leakage around the closure gate was encountered until the concrete reached such a height that its elevation was only slightly higher than the elevation of the water in the lake. Under this condition, pressure of the fresh concrete tended to force the gate away from the seals and permit leakage. Irregularity of the upstream steel forms also caused some difficulty. Pumping wells in the gate, which were designed but which were not installed, would have eliminated most of the undesirable features of the closure system.

Inspection.—Inspection of all operations in the production of aggregates: mixing, placing, and curing of concrete; sampling and testing of concrete; and special investigations in the dam were under the direction of the materials engineer. This arrangement permitted satisfactory adjustments or compromises in concrete mixes to fit best variations in concrete placing conditions.

During peak operations, as many as 27 inspectors and technical men were employed on various phases of concreting. During this time, crushing plant operations were on a 22-hour basis and concreting operations were on a 23-hour basis; each operation was covered by three shifts of inspectors. To provide for contingencies, such as illness, at least one man of all-round experience qualifications was held in reserve. When not required for relief duty, his services were used on work of routine nature, from which he could be immediately released in an emergency. Men normally assigned to special investigations were occasionally required for inspection duty as well.

Quarry, and crushing and screening plant inspection were handled by one man on each shift reporting to a mechanical engineer, who coordinated their findings and worked with the construction forces toward improved plant operation. Records furnished by the plant inspectors included screen analyses of all aggregates for use in designing concrete mixes and operating records of equipment such as hammer mills and screens. Their duties also included general inspection of quarry and plant, particularly the phases which affected quality of concrete aggregates.

An inspector was present during the placing of all concrete for permanent structures. Shift inspectors were responsible for general conditions during their respective shifts, and placing inspectors under the shift inspectors were responsible for individual pours. Placing inspectors were required to submit sketches of each concrete pour and to record data pertaining to the pour for future reference, including such items as dates and personnel. Shift inspectors inspected the clean-up operations after completion of concrete lifts and inspected forms and concrete surfaces prior to the starting of new concrete lifts. Inspection of curing of concrete surfaces was also made a part of the shift inspector's duties. A written statement covering operations during his shift was prepared by each shift inspector.

Inspection at the mixing plant was a divided responsibility. Two men were required, one a dispatcher and the other a general plant inspector. The extremely close attention to dispatching operations

required of one man in order to avoid confusion and delays, made it impossible for him to give but slight attention to details of plant operation. He was directly in touch with all concreting activities and with the characteristics of the concrete being produced, and consequently he assumed general responsibility. The plant inspector made moisture determinations on aggregates, calculated mixing water quantities, and made the routine plant inspections. The two men together prepared the shift concrete report.

Concrete inspection followed, in general, the usual relation between engineer and contractor except that more than ordinary efforts were made to promote unity of purpose. A spirit of cooperation between inspection and construction forces was fostered and proved to be successful in improving quality and in reducing the cost of concrete in place. It was understood by all concerned that quality of the work was of first importance. Questions of general policy were referred to the construction engineer. Other problems were usually settled on the job by the inspection and construction forces.

Three shifts of inspectors were maintained in the concrete laboratory. Each shift consisted of one inspector and one helper, whose duties were to sample concrete being placed, test the cured concrete specimens, and keep records of all work done in the laboratory. These men also did whatever experimental work on concrete mixes was required.

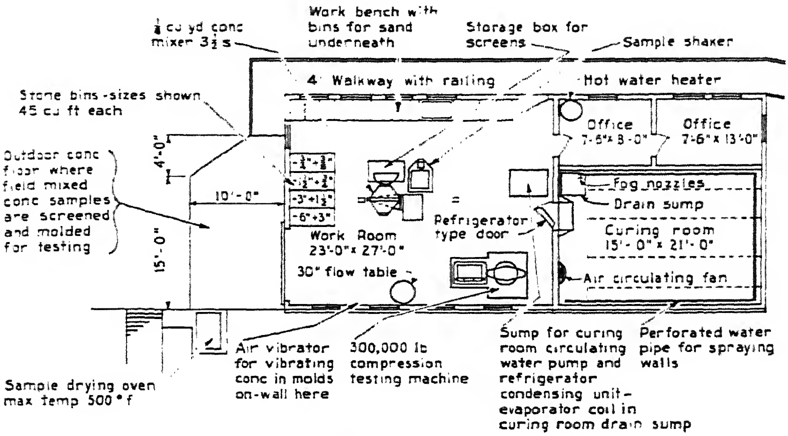
Special investigations, which required installation and operation of instruments for measurements such as temperature, strain, and deflection in the dam, were handled by one man of special qualifications assisted by one or two other men as conditions required.

Sampling and testing.

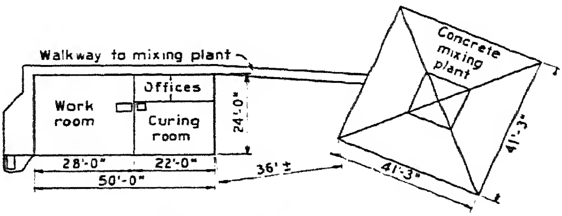
The concrete laboratory was located adjacent to the mixing plant for convenience in sampling field-mixed concrete. It was equipped for making usual record tests and for an appreciable amount of experimental work.

Samples of field-mixed concrete were obtained directly from the mixer discharge and were uniformly representative of the concrete being produced. The sampling operation required only a few seconds, and no delay was caused in concrete production. The sample container was dumped on a dampened portion of the concrete floor in the laboratory, and its contents, if of concrete containing larger than 1½-inch aggregate, was wet screened by hand through a 1½-inch square opening wire screen held at a slope of approximately 20 degrees from the horizontal and shaken vigorously. Samples containing maximum size aggregate of 1½ inches or less were not wet screened. Standard slump tests were made on all samples, and flow tests on a 30-inch flow table were occasionally included. After slump tests were made, the wet-screened samples were immediately made into 6- by 12-inch test cylinders. Concrete was compacted into the cylinder molds by means of an internal air-driven vibrator with flexible shaft and special vibrating element 1½ inches in diameter and 18 inches long. Each cylinder was vibrated for about 15 seconds to secure uniform compaction. In general, six test specimens were made from each concrete sample.

Cylinder molds were made from sections of pipe, machined inside and on both ends after the welding of lugs and the cutting of a narrow slit parallel to the axis of the pipe. Loosening of bolts permitted the slit to open and allowed removal of hardened specimens without difficulty. Machined plates, bolted securely to the cylinder molds, served as bases and provided one plane cylinder end which required no capping. Molds were cleaned in hot water and oiled with a mixture of machine oil and parawax applied hot with a paint brush.



ARRANGEMENT OF EQUIPMENT



GENERAL ARRANGEMENT

FIGURE 192.—*Field laboratory for testing concrete*

Except in extreme weather, test cylinders were allowed to remain in the laboratory for 24 hours or less, during which time caps of one to one sand-cement mortar were applied to the upper ends of specimens. Plane surfaces were obtained by use of plate-glass plates pressed against the mortar. Cylinder ends were cleaned and capped approximately 8 hours after the specimens were cast. Immediately after stripping, the cylinders were numbered and placed in a moist room maintained at 70° F. In extreme weather, test specimens were placed in the moist room immediately after they were made.

Test cylinders were broken in a 300,000-pound Southwark Emery hydraulic compression testing machine. The machine was calibrated by proving rings at the time of installation, August 1934, by Baldwin-Southwark's representative and again in July 1935 by the United States Bureau of Standards. The machine was furnished with three load-indicating dials of 0 to 30,000 pounds, 0 to 150,000 pounds and 0 to 300,000 pounds capacity. Two- by four-inch mortar cylinders and 6- by 12-inch and 8- by 16-inch concrete cylinders were tested. Only a few 8- by 16-inch cylinders were made because of insufficient capacity of the machine to handle them. A rate of loading of 17 pounds per square inch per second was used for all compression testing.

Test specimens were cured in a constant temperature room provided with storage shelves around the walls and in the center of the room. Walls and ceiling of the room were well insulated with mineral wool. Interior walls and ceiling were lined with galvanized sheet iron, having all joints filled with red lead. The floor was of concrete, sloping toward a sump located in one corner of the room. Water pumped from the sump and distributed by a perforated pipe suspended from the ceiling was allowed to run down the walls. A mercury regulator placed near the center of the room controlled heating or cooling of the circulating water to maintain a constant 70° F. air temperature. Fog nozzles provided 100 percent relative humidity. A fan kept air in circulation and prevented warm or cold pockets. Operation of the curing room was very satisfactory; its initial cost was not excessive, and maintenance was negligible.

Concrete test mixes were made in a small tilting mixer. A small priming batch always preceded the test batch in the mixer. Laboratory concrete, after mixing, was handled in the same manner as described for field-mixed concrete. Aggregates used in laboratory mixes were screened to the standard screen sizes above 200 mesh and stored in separate bins from which they were taken for recombining into the grading desired. Material passing the 200-mesh screen was considered as one size though it would have been desirable to have it in two or more sizes had the separations been feasible.

Behavior of dam during and after construction.

Investigations of temperatures, strains, contraction joint openings, deflections, hydraulic uplift, and other phenomena in the dam and foundation were undertaken during the construction and loading of the structure. Only a brief description of the program will be included as, to date, final studies have not been completed.

Temperatures at various locations in the dam are measured by Carlson resistance thermometers cast in the concrete or grouted into drill holes in the foundation. Three-conductor lead wires from each thermometer are brought to terminal boards at convenient locations in galleries of the dam. Resistance measurements were made by a Wheatstone bridge of special design connected to the thermometer leads for furnishing data for computing temperatures of thermometers and adjacent concrete. The smallest bridge reading corresponds to 0.1° F.

Strains in the concrete are measured by Carlson elastic wire strain meters embedded in the concrete. Two construction blocks of the dam are being investigated near the foundations for distribution of

stress. Three stations with 12 meters at each station were installed in each block. In each case meters were installed in duplicate. "No stress" meters were also installed to furnish data on length change due to such properties as moisture and temperature of unstressed concrete. The installations furnish data for computation of strain in a plane at right angles to the axis of the dam. Both three- and four-conductor lead wires were used for the strain meters, three-wire leads for meters to measure only strain, and four-wire leads for meters to measure both strain and temperature. With proper changes in connections, strain meters are read by the same Wheatstone bridge used for thermometers. The smallest reading on the bridge corresponds to four-millionths inch per inch of strain. Operation of the meter, as the name suggests, is based on the fact that the change in strain of an elastic wire is proportional to its change in resistance.

Contraction joint openings are being measured with elastic wire joint meters, similar in construction to the strain meters. A typical installation is shown in figure 34. One-half of the meter is cast in each block, separated by the joint to be measured, and change in length of the meter is used as a measure of joint opening. The meters have a range of approximately 1/4 inch and a least reading of about 0.0007 inch. Three- or four-conductor leads were used depending on whether joint opening only, or both joint openings and temperature were desired. The same Wheatstone bridge is used for joint meters as for thermometers and strain meters.

The method[17] used in placing and for reading the resistance thermometers, strain meters, and joint meters are similar to those used at Boulder Dam.

Deflection of block No. 35 is being measured by a 1/64-inch stainless steel wire plumb line suspended in a vertical shaft connecting the 1,051 gallery, the 880 gallery, and the 811 gallery. The top of the line at the 1,051 gallery remains fixed. Provision was made for the accurate reading of the plumb line at the 880 gallery and at the 811 gallery. A movable micrometer head microscope and fixed glass scales permit location of the plumb line to within 0.0005 inch. Horizontal measurements, both parallel and at right angles to the axis of the dam, are made. A weight attached to the free end of the plumb line is suspended in a small tank of kerosene to dampen vibrations. The line is entirely protected from external air drafts, except at the small openings used for making observations.

Results to date indicate that temperature deflections far overshadow the load deflections, change in deflection between winter and summer being at least 0.45 inch for the section of the dam investigated. During filling of the reservoir in the spring of 1936, the dam actually deflected upstream 0.3 inch due to temperature change.

Provisions were made for measurement of hydraulic uplift pressures at various locations at foundations and in the mass of the dam. Figure 35 shows details of cells used.

Temperature cracking in mass concrete.

Cracking of mass concrete, although not in excess of that in other comparable structures, has been more prevalent than the temperature

[17] Standford, Hollis, Field Procedure for Installing Embedded Instruments in Boulder Dam, U. S. Bureau of Reclamation Technical Memorandum No. 464.

rise in the mass concrete would seem to indicate. Due to favorable thermal properties of the dolomite aggregate, low cement content of the interior concrete, and other contributing factors, the average temperature rise in the mass was only about 35° F., somewhat less than in most large concrete masses not artificially cooled. No entirely satisfactory explanation of the inconsistency has yet been advanced.

At least half of all construction blocks cracked vertically at the faces halfway between contraction joints and horizontally at day's work joints at 10-lift (50-foot) intervals or less. Exposed contraction joint faces cracked vertically at re-entrant angles in keyways, cracks usually appearing in every other keyway, or about 20 feet apart. Depth of cracks varies greatly. Little is known about the extent of vertical cracks parallel to contraction joints, but absence of leakage from the lake to galleries through such cracks indicates that they are usually less than 20 feet in depth. Vertical cracks, parallel to the axis of the dam, extend completely across construction blocks at some locations. Horizontal cracks at day's work joints extend several feet in from exposed faces. In all cases noted, vertical cracks reduce to minute openings at foundations.

Low portions of construction blocks 46, 47, and part of 48 at the west end of the dam were placed with 7-day time intervals between successive 5-foot concrete lifts to minimize diagonal temperature cracking which was expected because of larger differences in elevation between sides of these blocks. This placing schedule reduced temperature rise in the concrete from approximately 35° to 24° F. The expected cracking was delayed until the concrete had cooled to within 10 degrees of its final temperature about 18 months after placing. Higher up in the same blocks, when the normal 3-day placing interval was resumed, the usual temperature cracking again appeared.

Block 38 at the elevation of the discharge conduits was constructed with an average time interval of a little more than 2 days between 5-foot concrete lifts in order to keep ahead of the two closure blocks 40 and 43. Block 38 had a high cement content (1.7 barrels per cubic yard) around the discharge conduits to provide concrete which could resist erosion. The high cement content and the short exposure interval for cooling produced an estimated temperature rise of 60° F. At a time of high block temperature, lake water was diverted through the discharge conduits, and concrete surface temperatures dropped rapidly. The resulting unfavorable conditions caused unusually severe cracking throughout the length of the tubes, vertical, horizontal, and diagonal cracks appearing on the surfaces. The extent of some of the cracks is shown by the fact that they carry moisture from contraction joints into the conduits and from one conduit to the other.

Horizontal cracking, usually at day's work joints, but occasionally between them, caused some leakage of reservoir water past the copper seals to the downstream face of the dam. Any open horizontal cracks extending deeper than the copper water seals across contraction joints near the upstream face of the dam provided paths for water to reach the contraction joints behind the seals. As the contraction

joints filled the leakage continued to the downstream face of the dam, bypassing the grout stops through horizontal cracks in a manner similar to that by which it gained entrance at the upstream face. During construction of the dam, some of the curing water found its way down the contraction joints and came out of horizontal cracks at faces of the dam. The fact that leakage to the downstream face is so small at present is evidence that either the cracks are closing appreciably or there is leakage past the horizontal grout stops across joints which separate grouting areas at 50-foot vertical intervals. Grouting of contraction joints should effectively seal off all leakage to the downstream face.

The proof has been positive that with the conditions under which the dam was built, the 3-day interval between successive 5-foot concrete lifts was insufficient to prevent undesirable temperature cracking in the mass of the dam. There are good indications that 7-day intervals between 5-foot lifts would have greatly reduced the cracking.

Factors controlling temperature cracking of concrete include variables such as thermal coefficients of expansion, changes in temperature, elastic modulus, tensile strength, and plastic flow. Characteristics of the cement and aggregate, proportioning of the mix, placing temperatures, weather conditions, and methods and schedules of concrete placement also influence this cracking.

With such an array of conflicting factors, complete control of temperature cracking seems to be in the distant future; however, experience gained from many large mass concrete structures and confirmed in general by observations at Norris Dam indicate that there are several construction procedures by which cracking may be materially reduced. Among them the following may be cited:

1. Use low concrete placing temperatures.
2. Use thin concrete lifts (5-foot or less) with suitable time intervals between lifts.
3. Keep the concrete placing schedule uniform.
4. Keep adjacent construction blocks at approximately equal elevations.
5. Cool the mass artificially.

The physical and chemical properties of the different types of cement undoubtedly have an effect on temperature cracking of concrete. Recent experience and study have shown that low-heat cement has rather definite advantages for use in mass concrete structures. However, at the time Norris Dam was started, the reasons for superiority of low-heat cement were not well known, and therefore type B cement was used. The type B cement was well suited for use in the dam, although in the light of present knowledge, the use of low-heat cement would have reduced the cracking in the dam.

Concrete costs.

Figure 193 shows a summarized analysis of the unit cost of mass concrete and the several materials and processes incident to the production of the concrete in place.

Table 91 shows the unit costs for the different classes of concrete for the dam and powerhouse.

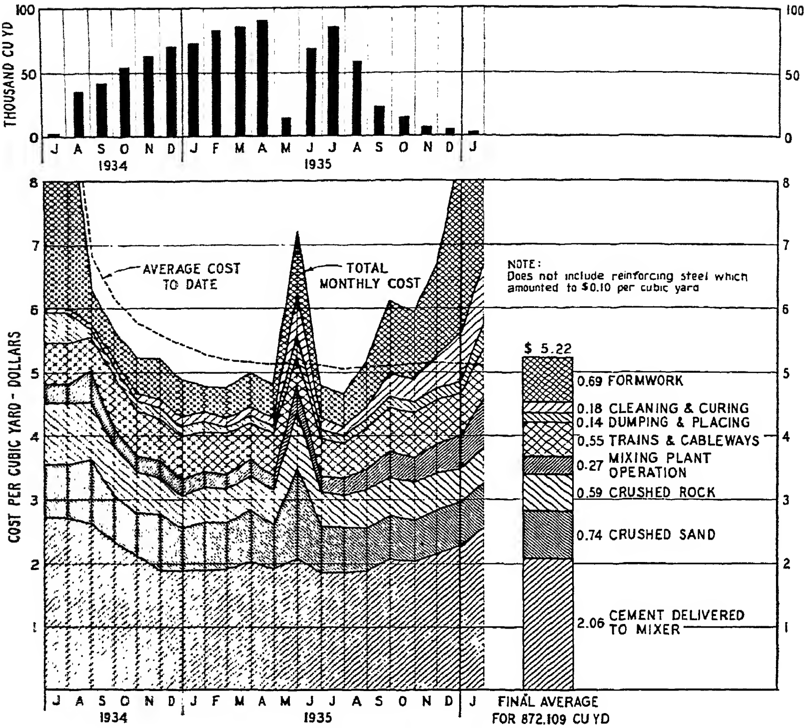


FIGURE 193.—*Mass concrete—Monthly quantities and costs*

TABLE 91.—*Unit costs for the various classes of concrete used*

	Cost per cubic yard
Mass	$5.32
Formed spillway crest and pier	14.04
Spillway apron	7.20
Reinforced training wall	13.44
Gravity training wall	6.81
Parapet walls and sidewalks	67.07
Penstock trashrack structure	54.16
Outlet trashrack structure	18.49
Powerhouse superstructure	59.18
Powerhouse substructure	15.07
East core wall concrete	11.38
Spillway bridge concrete	57.32

The above costs include reinforcing steel.

EAST EMBANKMENT

The eastern part of the dam is composed of an earth embankment section with a concrete core wall. The concrete core wall was constructed by excavating a trench to sound rock, parallel to the axis of

the dam. This trench was filled with concrete to form the core wall approximately 5 feet in width. It is 87 feet high for the first 180 feet from the dam. At this point there is a vertical step-up of 39 feet, and from here to the end of the wall, 584 feet from the dam, the height gradually diminishes until an elevation of 1,052 at the top of the wall is reached. The top of the core wall is at elevation 1,057 for the first 250 feet east of the end of the dam; at this point it is stepped down to elevation 1,052, which is the elevation of the top to the end.

A seam of unsound rock was uncovered at the bottom of the vertical step-up at elevation 965 and a tunnel was driven into the hillside following this seam in a position generally under the trench and extending a distance of 675 feet. Two shafts from the bottom of the trench to the seam served for access and removal of excavated material. Another tunnel, at the extreme east end of the cut, extended into the hill for about 65 feet at elevation 1,021.

Excavation.

Excavation for the trench was done largely by means of a dragline bucket on an endless cable operated from a stationary double-drum electric hoist. This equipment operated from two locations: the first, in a position to excavate the deeper portions of the trench nearest the dam, and the second from a position east of the extreme east end of the core trench to complete the excavation. In the first location a hopper on a tower was placed about 75 feet east of the tail-tower back track. The endless cable passed over a bent built above the hopper, and an inclined slide also resting on bents was provided for conveying the full dragline bucket to the hopper for dumping. This hopper discharged the excavated material into trucks which transferred it to the spoil area. After as much of the excavation as possible had been removed by this means, the rig was dismantled and moved to the second location. In this second location the hopper was not used, but the excavated material was moved to a point in front of the hoist by the dragline bucket. As the spoil built up it was moved out of the path of operations by a tractor equipped with a bulldozer. Since the trench passed through the fill for the cable-way tail-tower runway, special concrete beams were constructed to support the tail-tower tracks over the cut and permitted excavation without interruption of cableway operation.

The remainder of the earth excavation and all of the rock excavation was done by hand. It was hoisted to the surface in small mine cars which were raised and lowered by hoist-operated lifts. These lifts were located in such positions that shafts could be sunk from the bottom of the trench to the lower tunnel and could serve excavating operations at that point. The trench was double sheathed with 2-inch rough oak lumber and shored with 8- by 8-inch oak wales on 3-foot centers braced with 8- by 8-inch struts on 3-foot centers horizontally.

Earth excavation with the dragline rig started in May 1935, and was completed in August 1935. Hand excavation for the trench and tunnels was completed in November 1935. A total of 6,021 cubic yards of material was removed from the trench at a cost of

$17.29 per cubic yard, and 2,248 cubic yards of material were removed from the seam tunnel at a cost of $18.91 per cubic yard.

Tunnel.—The two shafts through which excavated material from the tunnel was removed were located 75 feet and 180 feet, respectively, from the west end of the tunnel. By extending the tunnel in both directions from each shaft and also by working from its west end, five faces were worked simultaneously. All material removed from the seam was loaded into the small mine cars and carried to the surface by the hoist-operated lifts. The main tunnel was approximately 6 feet wide and varied in height depending on the extent of the unsound material. Material from the small tunnel was disposed of in the same way as that from the large tunnel. All work in these tunnels was done by hand, and the walls were carefully cleaned prior to concreting.

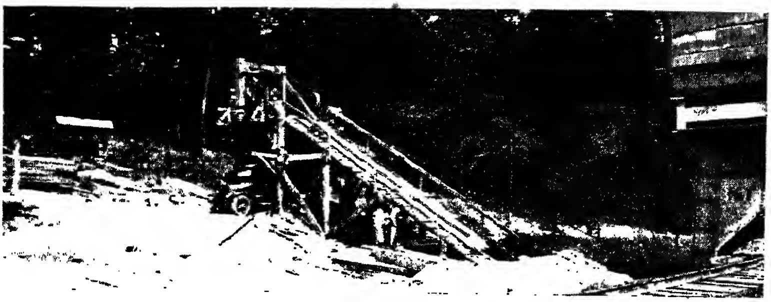


FIGURE 194.—*Hopper and slide used for loading trucks at east embankment.*

Concrete.

Mixes.—The core wall is composed largely of concrete containing 1½-inch maximum size aggregate with 1.50 barrels of cement per cubic yard and a water-cement ratio of 0.60 by weight. In a few instances, concrete was used containing 3-inch maximum size aggregate with 1.33 barrels of cement per cubic yard and a water-cement ratio of 0.55 by weight. In the tunnel section, a concrete containing ¾-inch maximum size aggregate with 1.75 barrels of cement per cubic yard and a water-cement ratio of 0.56 to 0.57 by weight was alternated with regular grout containing coarse sand as the largest aggregate with 2.80 barrels of cement per cubic yard and a water-cement ratio of 0.55 by weight. Concrete in the wall was consolidated by vibration with internal vibrators in the same manner as was concrete in the dam. In the tunnels, the concrete was spaded as much as possible. However, the close working quarters did not permit careful consolidation of the concrete as it was placed.

Core wall.—Concrete for the wall and tunnels was transported from the main mixing plant to the east side by regular transfer facilities. Buckets of concrete were dumped into a 12-cubic-yard Boulder-body dump truck equipped for dumping with a special manually

controlled end gate. Concrete was dumped from the truck into a hopper from where it was directed into the trench through wooden "elephant-trunk" chutes. Concreting of the wall was done in ten 50-foot blocks, each block separated by vertical contraction joints arranged with one vertical keyway and provided with a 16-gage copper water stop. Contraction joints extended to rock in one plane except in three instances where offsets were made in order to expedite the placing of concrete in the lower tunnel and to provide for slip joints. As a general rule depths of lifts were 5 feet, but in some cases lifts as deep as 10 feet were poured. Two blocks of the wall, between the east end of the gravity section of the dam and the tail-tower runways, were poured directly by cableway without additional handling.

The design and the construction departments both recommended that the core wall concrete be placed directly against the earth sides of the core wall trench as there was a possibility that there might be some voids in the earth back of the wood sheathing. To place the concrete against the earth sides of the trench, it was necessary to remove all cribbing and lagging as the concrete pouring progressed upward. Considerable difficulty was encountered in removing the lagging, as care had to be exercised against cave-ins and general crumbling of earth into the green concrete. Generally, short sections of studding were sawed off and the lagging removed plank by plank.

Concreting in the core trench was started in June 1935, and completed in January 1936. The total concrete placed in the core wall amounted to 5,988 cubic yards. It cost approximately $11.26 per cubic yard excluding overhead.

Tunnel.—Considerable difficulty was encountered in filling the two tunnels with concrete. A special hopper was used for filling the lower tunnel. Concrete was placed in the hopper and was subjected to the maximum air pressure obtainable from the plant air system. It was then discharged into an 8-inch pipe line leading to the point in the tunnel where the concrete was to be deposited. The pipe line became plugged frequently at first between a 90° bend at the intersection of the shaft and tunnel and the discharge end of the line. This was partially remedied by adding another air inlet at the bend, which aided in keeping the horizontal section of the pipe clean. The upper tunnel was filled in much the same manner except that gravity alone was depended upon to force the concrete through the pipe. Introduction of batches of very rich grout at rather close intervals expedited the movement of concrete through the pipes.

Concreting of the lower tunnel was done in sections approximately 75 feet long, while the upper tunnel, which was only 70 feet long, was concreted in one continuous pour. Provisions were made in the top of the lower tunnel for grouting any spaces left during the concreting process or resulting from shrinkage. A system of 1-inch pipes was arranged to extend into jackhammer holes 1 foot deep drilled in the top of the tunnel at high points. These were connected to two 1½-

inch headers which extended horizontally along the top of the tunnel and to the surface through the shafts. This system of piping also served to relieve air trapped during concreting.

Earth fill.

The earth fill consisted of clay obtained partly from borrow pits located about 400 feet upstream from the dam, and partly from the excavation for the approach roadway. Three different borrow pits were opened in an attempt to find material with proper moisture content. Test samples were taken to determine the suitability of borrow material and from the fill to determine the degree of compaction obtained. Laboratory tests indicated that a moisture percentage of about 20 to 22 percent would give maximum compaction. Samples

FIGURE 195.—*Material being placed in the downstream fill. The fill is about 75 percent complete.*

from the borrow pit and fill showed moisture contents ranging from 27 to 32 percent. The size and importance of the fill did not justify the equipment and procedure necessary for controlling the moisture content close to the optimum, therefore the testing done was merely a check on the compaction obtained rather than a control measure. Tests indicated that the moisture content of the fill in place was from 5 to 11 percent too high to obtain maximum compaction, and that best compaction was not secured with the prevailing moisture content.

Top soil and vegetation were stripped from the area of contact between original ground and fill by the 1¼-cubic-yard shovel, and spoil was moved to the waste dumps by a fleet of from 10 to 15 rented 4-cubic-yard trucks. The shovel opened the first borrow pit about 400 feet upstream from the dam in July 1935, after completion of the stripping. Soon after the opening of the first borrow pit the 1¼-cubic-yard shovel was replaced by a 3-cubic-yard shovel which

completed the borrow for the fill. Material was transported from borrow to the fill by rented 4-cubic-yard trucks and TVA-owned 8-cubic-yard and 12-cubic-yard trucks.

Material for the fill was dumped at the approximate point of use and spread by a bulldozer-equipped tractor into layers whose compacted thickness did not exceed 6 inches. At first, compaction was by means of a sheepsfoot roller. The first fill was placed immediately east of the abutment section, and excessive moisture in the clay in this location caused the sheepsfoot roller to clog considerably. Satisfactory compaction could be obtained by the use of loaded dump trucks equipped with dual tires; therefore, this method of compaction was chosen. An 8-cubic-yard truck loaded with material was tried but proved to be too light for proper compaction. A loaded 12-cubic-yard

FIGURE 196.—*Completed riprap on the upstream slope.*

truck was found to be satisfactory and was used throughout the job. In inaccessible corners and adjacent to the core wall it was necessary to compact the clay with hand tampers.

The fill was completed early in December 1935 and entailed the placing of 71,608 cubic yards of material, approximately 85 percent of which was obtained from borrow pits and the remainder of which was excavated from the east approach roadway. Final unit costs for the rolled fill, excluding overhead, amounted to $0.77 per cubic yard.

Fill protection.

To protect the upstream side of the fill a layer of rock riprap laid on a gravel blanket was used. A footing for the riprap is provided at the toe by a trench 4 feet wide and between 2 and 3 feet deep. The blanket consisted of ¾- to 1½-inch crushed stone spread to a thickness of 1 foot over the entire surface of the fill. Stone from the aggregate quarry, ranging in size from ½ cubic foot to 2 cubic yards and placed to conform approximately to the shape of the surface of

the earth fill, completed the riprap. This material was handled from the west bank to the fill by cableway. The blanket material was handled through the mixing plant and by the concrete transfer cars and concrete buckets. Crushed rock for the blanket was spread as much as possible while dumping. Areas inaccessible to the cableway were covered by spreading with a tractor and bulldozer. Placing of the larger stone was first done by hand but later a special skid derrick was constructed and used. The downstream side of the fill was protected against erosion by a blanket of lespedeza grass.

The placing of riprap was done between November 1935 and February 1936 at a cost, excluding overhead, of $0.913 per square foot of surface, or 4,004 cubic yards at approximately $7.35 per cubic yard. A total of 1,155 tons of crushed rock and sand was used for the blanket and 6,856 tons of derrick rock for the surface. The total area covered amounted to 32,200 square feet.

PENSTOCKS

All material for the two plate-steel penstocks was furnished, fabricated, and installed by contract.

Three bids received in response to the Authority's invitation were as follows:

Chicago Bridge & Iron Co.	$87,500
Bidder No. 2	183,542
Bidder No. 3	195,238

Because of the great difference between the bid price of the low bidder and the other two bidders, considerable investigation was done by the Authority before awarding the contract. However, the low bidder was found to be responsible, both from the standpoint of financial strength and experience on comparable hydraulic structures, although they had had no previous experience on class I welding. The question of whether the code under the N. R. A. covering this class of work would be violated by the low bidder was also advanced by one of the other bidders. The N. R. A. administrator ruled that the low bidder would not be guilty of code violation, and the award was subsequently made to the low bidder.

Contractor's plant.

A field fabricating plant was set up by the contractor on a flat field adjacent to the left bank of the river and approximately 1,500 feet downstream from the dam. One guy derrick, operated by a double-tandem friction drop hoist, driven by a 100-horsepower electric motor, constituted the main piece of handling equipment. The remainder of the plant and equipment was arranged around this hoist within the handling radius of the derrick boom. This derrick was capable of handling 40 tons at 50-foot radius and 20 tons at 75-foot boom radius, and the hoist had a line pull of 10,000 pounds with 800 feet of cable on the drum. Twenty 400-ampere welders, one 10-foot and two 20-foot roller beds, and the necessary additional equipment for welding, stress relieving, and X-raying operations made up the remainder of the plant. A bank of three 500-kilovolt-amperes 2,300/440-volt transformers rented to the contractor by the Authority was installed at the fabricating yard to provide power

for heating in connection with stress relieving. Lighting transformers as well as electric power for all purposes were furnished at predetermined prices by the Authority.

Shop joint-welding procedure.

The plates for the penstocks were rolled into half rings, 20 feet in diameter by 10 feet long, and the edges planed for welding at the Birmingham, Ala., plant of the contractor. They were shipped to Coal Creek, Tenn., in this shape and hauled to the field fabricating plant by truck trailer. At the field fabricating plant, they were unloaded onto an assembling or forming fixture, where two matched halves were assembled into a ring 20 feet in diameter and 10 feet long and tack welded together with a small weld bead for the full length of each longitudinal seam. The general procedure was to weld the two longitudinal seams of each 10-foot section and then join two 10-foot sections, making a section 20 feet long and 20 feet in diameter. Sections of this size were transported to the site for placement in final position.

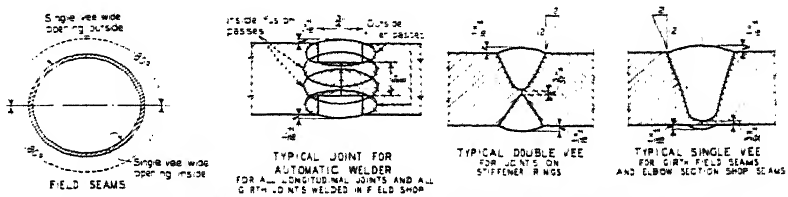


FIGURE 197.—*Details of various types of welds used in Norris penstocks.*

Ten-foot sections.—The first operation after longitudinal seams had been tack welded was to tack weld three 2-inch pipes inside the section to act as braces in maintaining the assembled 20-foot pipe in a practically circular shape prior to the placing of the stiffener rings and to supplement the stiffener rings in holding the pipes circular during the transporting and placing process. The assembled 10-foot pipe section was then placed on a 10-foot roller bed with one longitudinal joint at the bottom.

For this part of the operation the roller bed was inclined 4° with the horizontal so that one edge of the penstock pipe section, when in position on the bed, was slightly lower than the opposite edge. Welding of the longitudinal joints with the automatic welder was started in each case at the lower side of the section. Complete welding of the two longitudinal joints of a 10-foot section of pipe was done as follows:

1. Starting and finishing plates were tacked to each end of the seam. These plates were also used as test specimens for physical tests on the weld.
2. One fusion pass by the automatic welder was made on the inside of the longitudinal seam at the bottom of the pipe, and one fusion pass by the automatic welder was made of the outside of the longitudinal seam at the top of the pipe.
3. The pipe ring was then rolled 180° on the roller bed, reversing the original position of the two longitudinal joints.

4. One fusion weld was then made on the outside of the longitudinal seam at the top, and one fusion pass on the inside of the longitudinal seam at the bottom of the pipe. The four fusion passes just described, one on the outside and one on the inside of each longitudinal seam, were made on each ring.

5. The pipe ring was then rolled 180° on the roller bed. The first two fusion passes were cleaned of all slag and a bar of proper thickness to suit the thickness of the plate of approximately the same analysis as the plate was tacked in place in each groove and one filler pass made on each seam, first on the inside of the bottom seam and then on the outside of the top seam.

6. The pipe ring was then rolled 180° and the process described in (5) repeated but reversed for the remaining two grooves, thus completing the welding of longitudinal joints for a 10-foot section of pipe. The same procedure was followed in welding the longitudinal joint of an elbow section of each pipe. All longitudinal welds were made by automatic welding machines. The following table shows the characteristics of operation for the two types of passes on a longitudinal seam:

Type of pass	Speed of travel	Arc voltage	Current	Flux
Fusion	3.8 inches-4 inches per minute	35-37	525-550	Suitable low speed.
Filler	do	33-34	650	High speed.

Twenty-foot sections.—As longitudinal seam welds were completed, each 10-foot section was placed on a 20-foot roller bed for welding at the circumferential joint to its companion 10-foot section. Two 20-foot roller beds were provided, one for welding inside of the seam and one for welding outside of the seam. The welding of the circumferential joint required the following steps:

1. Two companion 10-foot sections were placed on the 20-foot roller bed provided for welding the inside of the circumferential seam.

2. One fusion pass was then made on the entire circumference of the seam, after which the seam was thoroughly cleaned of all slag. A bar of the proper size and of approximately the same analysis as the plate was tacked in place and one filler pass made, completing the welding of the inside of the circumferential joint.

3. The 20-foot section of pipe was then moved to the roller bed provided for welding the outside of the circumferential weld.

4. One fusion weld was made on the entire outside circumference of the seam, after which the seam was thoroughly cleaned of slag. A bar of the proper size to suit the thickness of the metal was tacked in place and one filler pass completed the welding of the circumferential joint, making two 10-foot sections into one 20-foot section.

The two 20-foot roller beds were equipped with electrically driven roller mechanisms to rotate the pipe at a peripheral speed of 4 inches per minute, which coresponded to the speed of travel at which the welding machines were operated on the longitudinal seams. All shop circumferential seams were welded by automatic welding machines mounted on swinging jibs which were attached to vertical masts, one

for each of the roller beds. The roller beds for welding the longitudinal seams on the 10-foot sections were provided with an elevator for moving the automatic welder to the top of the pipes for welding the outside of the longitudinal seams. The only shop seams not welded by automatic welding machines were the circumferential joints which connected the two pieces forming the elbow section for the downstream end of each penstock. These joints were hand welded in the field fabricating yard, using the same procedure as that for field hand-welded circumferential joints.

Stiffener rings were not welded to the pipes according to any set schedule. In general, stiffener rings were welded some time during the process of welding the circumferential seams. At times, stiffener rings were welded on the 10-foot section as soon as it was placed on the 20-foot roller bed before the companion section was ready for welding of the circumferential seam.

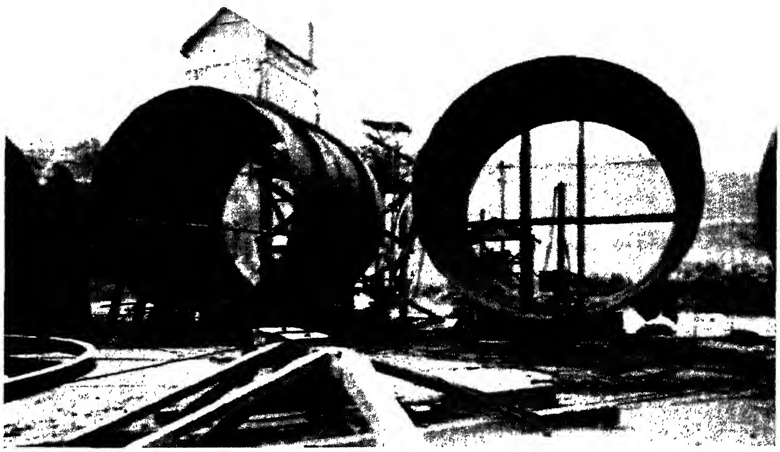


FIGURE 198.—*Field welding in progress.*

The scarf for the automatic welding machine was $\frac{3}{4}$-inch wide and varied in depth with the thickness of the plate, so that the thickness of the center section of the scarf was always $\frac{5}{8}$ inch. This assured a uniform treatment in making fusion passes on automatically-welded joints.

X-raying shop joints.

After completion of all welds in each 20-foot section and each elbow section of pipe, the welds were examined by a General Electric X-ray Corporation, type KXC–3 shock-proof, 200 kilovolt peak, 10 milliampere X-ray unit. Mounting of the shock-proof tube of this type unit was on a tripod and connected to the high-tension generating unit by two 50-foot cables to permit flexibility. To protect

the operator, the control stand was mounted behind the high-tension generating unit in a lead-lined cabinet. Mounting of the X-ray tube on the tripod was such that the target was 120 inches from the inside surface of the seam being radiographed and the top of the tripod was arranged so that the tube could be transited through 360°, thereby radiographing an entire circumferential seam from one position in 24 exposures.

Longitudinal seams were radiographed by moving the tripod on rails laid in the bottom of the pipe. Four separate settings were necessary to radiograph the longitudinal seams since the two 10-foot sections of pipe were joined together with the longitudinal seams in one 10-foot section placed 90° from the two seams in the other 10-foot section. Four exposures were necessary to cover each 10-foot section of longitudinal seam, making a total of 40 exposures for each 20-foot section of shop fabricated pipe.

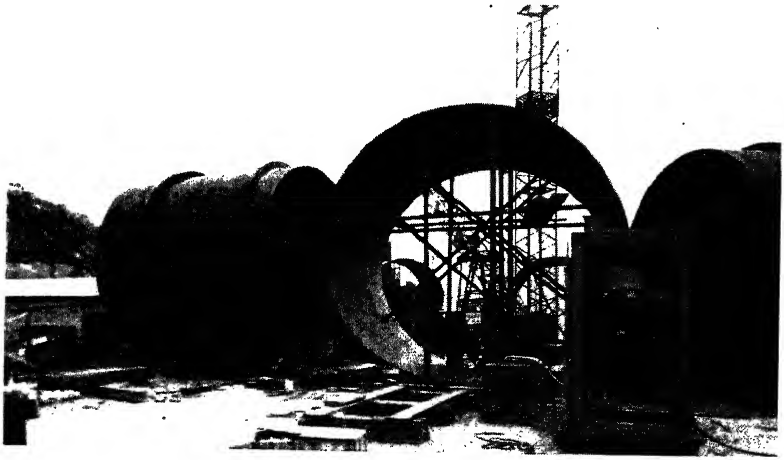


FIGURE 199.—*X-ray operations.*

After X-raying, the films were examined by TVA inspectors and the parts not acceptable after comparison with standard acceptable pictures were rejected, chipped out, and rewelded. After rewelding, the sections were again X-rayed and either rejected or accepted. After acceptance, the section of pipe was moved to the stress relieving frame.

Stress relieving shop joints.

A structural steel frame built to support a single 20-foot section of pipe was provided on which the stress relieving was done. The frame was provided with a structural steel ring, which supported heater pads for stress relieving the circumferential seam. The frame was separated at the top and hinged on each side, so that the ring could be opened like a clam shell to permit placing the 20-foot section on the frame. Attached to the ring were four structural

steel arms, two extending laterally from each side of the ring to support the heater pads for stress relieving the longitudinal seams. All longitudinal seams and all shop circumferential seams were stress relieved[18] by the heat applications. A band extending not less than six times the thickness of the plate on each side of the seam was treated. The electric heating pads for the circumferential seams had a heating face 20 inches wide by 18 inches long and for the longitudinal seams 24 inches wide by 18 inches long. Each heater had a rated capacity of 19 kilowatts. Half-size heaters, 20 inches wide by 9 inches long and 24 inches wide by 9 inches long, each with a rated capacity of eight kilowatts, were also used. The connected load for stress relieving one section of pipe 20 feet long was approximately 1,200 kilowatts. All heaters were placed on the outside of the pipe, and the inside of the pipe was heavily insulated by mineral wool blankets in the area being heated. These were held in place by a light structural steel framework built to conform to

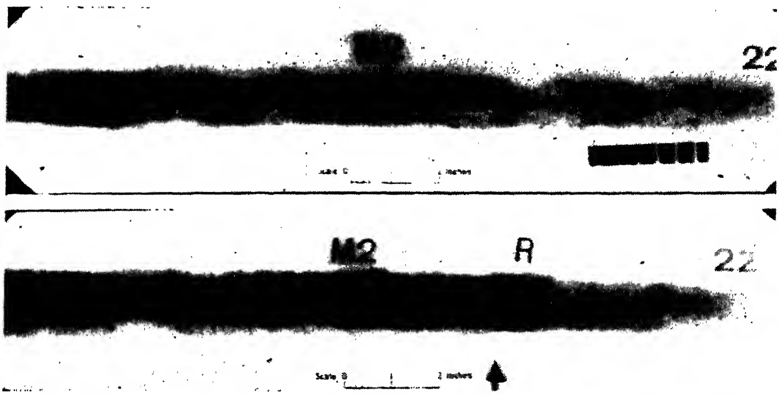


FIGURE 200.—*Typical X-ray picture of rejected (above) and acceptable (below) welds.*

the shape of the pipe and to match the outside framework which supported the heaters. Asbestos rope was packed around heater units to prevent loss of heat during the stress relieving process.

The switchboard, from which the heaters were operated, was arranged for either manual or automatic control. The heating system was made in 10 separate circuits with the heaters for a circumferential joint being divided into 6 separate circuits, and 1 individual circuit being provided for each 10-foot longitudinal joint. Automatic control was effected through the action of 10 chromel alumel thermocouples, one provided for each of the 10 heater circuits. An automatic switch cut in the thermocouples one at a time and registered the temperature at the point of the thermocouples. The same switch then cut the heater circuit on or off, depending on whether the temperature was too high or too low. This control also included a time-control switch to insure a uniform taking on or release of load.

[18] In accordance with the American Society of Mechanical Engineers Boiler Construction Code, Unfired Pressure Vessels Section.

The time-control switch was operated by 2 timers which cut in the power on all 10 circuits at the rate of 1 each 2 seconds.

Temperature control, however, took precedence over the time-control switch in the automatic operation of the heater circuits. The automatic control proved satisfactory up to about 800° F., but above this temperature manual control had to be used throughout the remainder of the temperature rise and also throughout the entire holding period. As a check on the temperatures during the stress-relieving process, 30 iron-constantan thermocouples, spaced uniformly along the seams being stress relieved, were provided and connected through a selector switch to an indicating pyrometer calibrated to read directly in degrees Fahrenheit. These thermocouples were

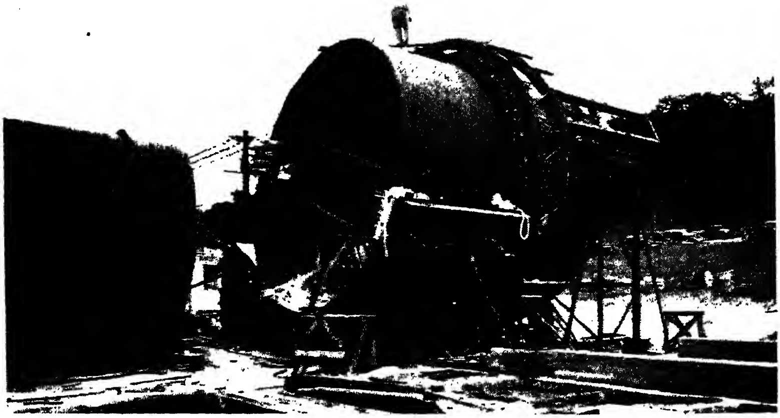


FIGURE 201.—*Heating units in place on a 20-foot pipe section.*

placed on the inside surface of the pipe. Preliminary tests were run with thermocouples placed on both inside and outside of the pipe and with temperatures between 1,100° and 1,200° F.; the difference in temperature from one side of the plate to the other did not exceed 20° F.

The stress-relieving process consisted of bringing a band as wide as six times the thickness of the plate on each side of the seam being stress relieved up to a temperature of about 1,200° F. and holding this temperature for at least 1 hour for each inch of thickness of plate. The time in which the desired temperature was to be reached after heating started was not specified in the code. In general, about 3 hours were required to bring the pipe to the desired temperature, depending to a large extent on the outside temperature and the efficiency of the insulation. The thickest plate required a theoretical holding time of 1 hour and 22½ minutes. However, the holding time actually ranged between about 1 hour and 30 minutes to 1 hour and 55 minutes. After the holding period was over, the pipe was allowed to cool with the heaters and inside insulation in place. The cooling process usually required 11 to 12 hours.

Transporting and placing pipe sections.

Completed sections of pipe were loaded by the contractor's derrick onto a 60-ton float trailer owned and operated by the Authority, and transported to a point under the two cableways for handling into final position in the dam. The trailer was pulled by two 75-horsepower tractors. Structural steel cradles to support the pipe in place and anchor bars to tie the pipe down were furnished and placed by the contractor, except that anchor bolts and anchor bars were set to line and grade by the Authority. The first section of pipe completed and moved to final position in the dam was the elbow section for the west penstock. After the elbow section was placed, each succeeding section proceeding toward the upstream end of the pipe was placed. Both cableways, lifting together, were used to transfer the individual sections of pipe from the trailer to the cradles in the dam. The heaviest section of pipe handled weighed approximately 36 tons. As a general rule, sections of pipe were transported and placed on Sunday, when the cableways were released from concreting. The contractor was charged rental on the trailer and tractors used in transporting the penstock sections from the fabricating yard to the cableways, but no charge was made for cableway use.

Welding field joints.

All field circumferential joints were welded[19] by hand. The top half of each field seam was made by hand welding, a single V placed on the outside of the pipe and the bottom half by welding a single V placed on the inside. The method followed was that of "laying in" shallow weld beads by the cross-weave procedure and carefully peening each layer before the next bead was laid in.

Sections of pipe when placed in their final position on the cradles were supported entirely by steel struts. These struts remained in place until the joint had been properly matched and secured by short steel straps which spanned the joint and were tack welded to the two sections. The bracing was then removed and the welding started on the inside of the pipe at the bottom and was carried both ways simultaneously to the horizontal center line, working about 18 inches at a time on each side. After completion of this section, welding was started on the outside of the pipe at the horizontal center line, working both sides simultaneously toward the top in approximately 18-inch lengths. When the main weld was completed, the plate at the base of the V (figure 197) was chipped out from the opposite side of the plate from which the main weld was made and the resulting notch filled in the same manner as the V of the main weld. The metal forming the weld was in all cases deposited in horizontal layers, starting at the bottom of the V. After each layer was deposited it was carefully peened before the next was deposited; and each succeeding layer was handled in the same manner until a weld approximately 18 inches long was completed. The procedure was then repeated on each adjoining 18 inches of the seam progressively, until the outside welds met at the top, or the inside welds each reached the horizontal center line. Approximately 12 passes were necessary to build the main weld from the bottom

[19] In accordance with the American Society of Mechanical Engineers Boiler Construction Code, Unfired Pressure Vessels Section.

of the V to the top, and two passes were necessary for the small weld on the opposite side of the plate from the main weld. The same procedure was used in making the shop hand-welded circumferential seam which joined together the two sections of pipe forming the penstock elbow. Only one hand-welded shop joint was made for each penstock.

X-raying field joints.

The procedure for X-raying the field-welded circumferential seams was identical with that used for X-raying the shop-welded circumferential seams. A total of 24 exposures was necessary to radiograph completely a circumferential joint.

Stress relieving field joints.

A decision was reached, after one field seam had been stress relieved, to abandon stress relieving on the remainder of the field circumferential joints. This decision was based on the theory, which was substantially confirmed by test later, that in stress relieving the field circumferential joints, residual stresses were set up which were greater than those left by the welding process. Two test pieces, one from the stress-relieved joint and one from a joint not stress relieved, were removed from the pipes in order to determine the direction and magnitude of the residual stresses. The comparison of these two pieces substantially confirmed the original theory.

Tests on stress relieving.

Tests on one of the 20-foot welded steel penstocks in place were made for the purpose of determining stresses introduced during fabrication. The particular purpose of the tests was to determine the

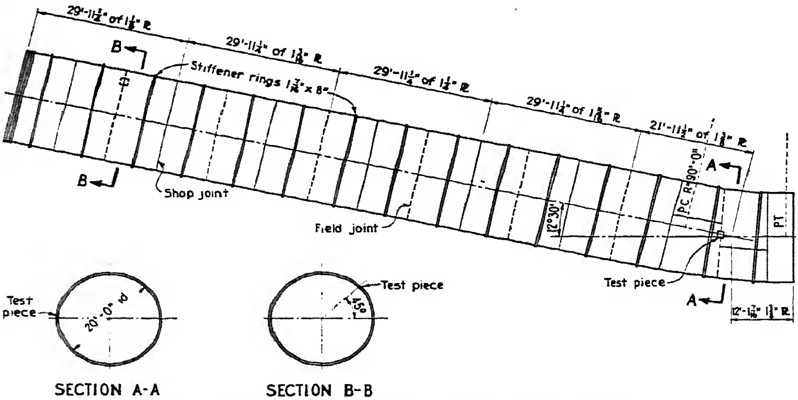


FIGURE 202.—*Location of test pieces in west penstock.*

effectiveness of the stress-relieving process when applied to the field-welded circumferential joints. Strain gage measurements were made in each of two 10-inch squares, one across a stress-relieved joint and one across a joint which had not been stress relieved.

All longitudinal joints and the circumferential "shop" joints were welded by carbon arc and were stress relieved at a temporary assembly plant near the dam. The pipe sections thus made up were placed in their final positions in the dam and hand welded by a shielded arc. The field joint at the location of the lower test piece was stress relieved, but the remaining field joints were not. All internal bracing was removed before the tests. The pipe was supported by steel cradles between field joints.

For the tests the gage lines were measured with a 10-inch Whittemore strain gage graduated to 0.0001 inch and a 2-inch Olsen strain gage graduated to 0.00006 inch. An invar standard bar was used

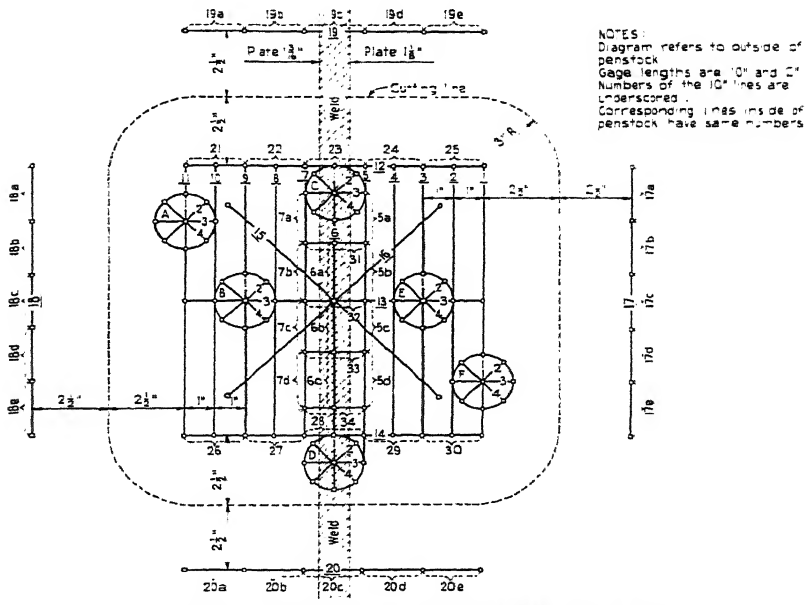


FIGURE 203.—*Gage line diagram—Unrelieved penstock joint.*

with the 10-inch gage and a mild steel bar with the 2-inch gage. The field measurements were taken in the early morning in order to minimize temperature differences between the inside and outside of the pipe. The test piece was then removed by close drilling ⅜-inch holes on the line marked "cutting line". Vertical lines were drilled first, one hole next to the preceding hole. Horizontal lines were drilled in such a manner that the amount of remaining metal per unit length was nearly constant. After removal, the gage lines on the test piece and the remaining gage lines on the pipe were again measured. The test pieces after removal were measured in a constant temperature room at 70° F., and all field measurements were reduced to 70° F. for comparison.

In the case of the upper test piece, each gage line was measured from two to five times, both in place and after removal, by two observers. The average probable errors, as determined by discrepancies in the measurements of each line, were as follows:

Location	Average probable errors in inch per inch	
	10-inch lines	2-inch lines
Inside	±0. 00001	±0. 00006
Outside	±. 00001	±. 00003

The maximum probable errors were ±0.00005-inch per inch for the 10-inch lines and ±0.00015-inch per inch for the 2-inch lines. Another check on the accuracy was furnished by division of some of the 10-inch lines into 2-inch segments. The average difference in strain as indicated by the full line and by the 2-inch segments was ±0.00007-inch per inch. The average discrepancy in the measurement of the 2-inch rosettes as indicated by a comparison of the diagonal lines with the horizontal and vertical lines was 0.00004-inch per inch. Errors of measurement on the stress-relieved test piece were probably somewhat greater since fewer readings were taken.

The plotted results of the measurements are shown in figure 204. In plotting the deformations, it has been assumed that the vertical lines deformed symmetrically with respect to the horizontal center line, and that the horizontal lines deformed symmetrically with respect to the vertical center line.

The measurements confirmed doubts as to the value of this method of stress relieving applied to the circumferential field joints, and consequently the process was discontinued after a trial on one joint. It was concluded that when the joint was relieved of stress by heating a limited area to the temperature at which the metal became plastic, the subsequent cooling, since the area was restrained from free contraction in the circumferential direction, introduced circumferential tensile stress. The longitudinal bending stress shown by the stress-relieved test piece is attributed to more rapid cooling of the outside of the pipe than of the inside after the heaters had been shut off.

Cleaning and painting.

After all field seams had been welded, the inside surface of each penstock was thoroughly cleaned by means of a portable sand blast machine, using steel grit as the abrasive. Sections of pipe approximately 20 feet long were isolated by tarpaulin bulkheads and completely cleaned of all rust and other foreign matter. Cleaned sections were given one coat of bituminous primer by spray and brush.

After the contract was completed and the pipes accepted, the pipes were again thoroughly cleaned and a coat of bitumastic enamel placed over the entire inner surface. This work was done by the forces of the Authority.

Inspection and testing.

The Authority maintained a force of three inspectors in addition to one inspector loaned by the Bureau of Reclamation who was present during the beginning of the field fabrication procedure. Inspection

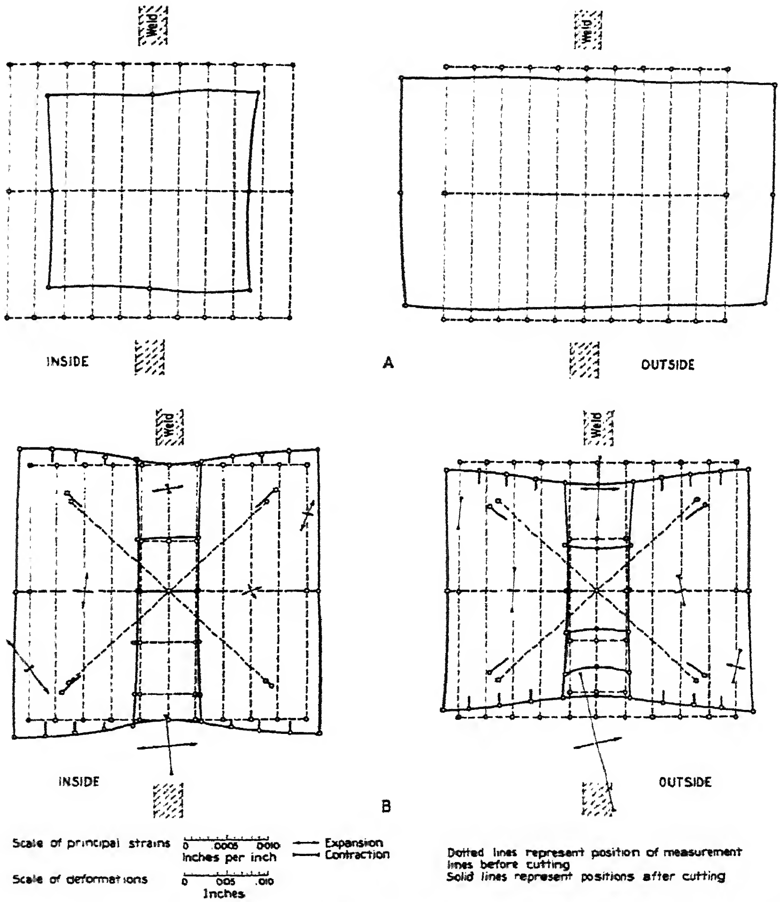


FIGURE 204.—*A. Deformations in test piece removed from welded penstock which had been stress relieved. B. Deformations and principal strains in test piece removed from welded penstock joint which had not been stress relieved.*

was carried on during three 8-hour shifts with one inspector on duty during each 8-hour shift.

Each welder was required to qualify in the presence of a TVA inspector for each type of welding he was to do. Qualification included going through the entire process of welding a seam of the

same material as the plate in the pipes under circumstances simulating actual conditions.

All X-ray pictures were made in the presence of a TVA inspector. Each exposure was identified by lead reference markers which were placed directly over permanent identification marks stamped on the parent metal. Two penetrometers with grooves representing 2 percent of the plate thickness were placed near the welds, one at each end of each exposure. X-ray film, 4½ inches wide by 34 inches long, was used for the pictures. The films were enclosed in a casset, having superspeed intensifying screens. When loaded, the casset was placed in an open-face lead-lined box and securely fastened, face down, over the seam to be radiographed. After several experimental exposures, it was determined that the required time for best film exposure was 4 minutes for 1⅜-inch plate with a primary line voltage of 168 volts. Each film overlapped the preceding film two inches to insure complete coverage. In making reshots of repaired sections, the reshot film carried the same identifying number, with the addition of R, as did the original film. In cases where more than one reshot was necessary at one location, only the original and the acceptable films were filed, all intermediate reshots having been destroyed. All films were either accepted or rejected on the basis of a comparison to the sample pictures of acceptable welds. On the average, one chip-out and reweld was necessary for each 10 feet of welded seam.

In addition to the X-ray test, a series of four physical tests was conducted on specimens made from the starting and finishing plates for the longitudinal seams. These tests included free bend, reduced tension, all-weld tension, and specific gravity. All laboratory tests were made in the presence of a TVA inspector.

Value of stress relieving.

The value of stress relieving of circumferential field joints by heating the weld and a limited area on each side of it while maintaining the remainder of the metal at air temperature was considered questionable. To determine the effect in an actual case, tests were made on the first stress-relieved field joint to determine the state of stress at the weld. Ten-inch strain gage patterns of 20 lines each were laid out on the inside and the outside of the pipe, centered on the weld, and designed to measure longitudinal, circumferential, and diagonal strains. Initial measurements were made after stress relieving and the test piece was then removed by close drilling a 15-inch square. A comparison of measurements before and after removal of the test piece indicated the magnitude of the residual stresses, assuming that after removal the test piece was free of stress. The process was then repeated on a joint which had not been stress relieved. The procedure was the same as before, except that in the second case both 10-inch and 2-inch gages were used, and each pattern consisted of 91 lines. A total of over 1,200 strain gage readings was taken. The measurements indicated that stress relieving of the field joints could not be accomplished by heating only the weld and a short section of pipe on each side of it.

Contractor's organization.

The penstock assembly and erection were under the direct supervision of the chief engineer of the contracting company. In addition, the supervisory personnel included one engineer, one superintendent, two shift foremen, and the following men:

Number of men	*Classification*
1	X-ray photographer
1	Derrick operator
1	Storekeeper
25 to 35	Welders
25 to 35	Chippers and peeners

Job progress.

Award of the contract was made on June 15, 1934, and about 30 days later the first material for the pipes was delivered to the field fabrication plant. The first section of pipe was placed in final position on September 9, 1934, and the work of the contractor was completed on November 3, 1934, 141 days from the award of contract.

POWERHOUSE

Construction of the powerhouse was divided into two phases: the substructure which consisted mainly of concrete surrounding the draft tubes and scroll cases; and the superstructure including the structural steel frame and concrete for the powerhouse walls. Since the speed rings and scroll cases were erected prior to concreting of a part of the substructure, details of their erection will be described here, and the other items of permanent powerhouse equipment will be treated later.

Substructure.

Draft tubes.—Forms for the draft tubes were fabricated completely on the carpenter shop platforms and moved by cableway directly into final position in the powerhouse. Ribs for the forms were of double 2- by 10-inch timbers spaced 16 inches on centers. The downstream 20 feet of each tube form was lagged with 2-inch pine while the remainder, in order more easily to form the curved surfaces of the elbow section, was lagged with a double layer of narrow 3/4-inch-thick strips.

Each form was cut into five sections to facilitate handling and placing. The three horizontal arms and the elbow and throat sections made up the five pieces. A single cableway placed each section separately, beginning with the horizontal arms, then the elbow, and finally the throat section.

The usual and logical procedure in setting draft tube forms is first to place the pier nose castings and then concrete the base of each casting before placing the forms. Shipment of the pier nose castings was delayed several weeks; therefore, the usual procedure was reversed to avoid any delay in the preparation of forms for that part of the powerhouse in which the draft tubes were located. This necessitated removing part of several ribs and some lagging on each side of each form in order to clear the bottom flanges of the castings as they were lowered into place.

Concreting of the substructure around the draft tubes was carried out in eight stages, using a mix composed of 1½-inch maximum size aggregate with 1.50 barrels of cement per cubic yard and a water-cement ratio of 0.55 by weight. In sections not heavily reinforced, and where the form was comparatively large, some mass concrete was used. Mixing and placing was done by the regular mixing and transporting facilities.

FIGURE 205.—*Draft tube forms in place.*

The draft tube liners were included in the turbine contract and furnished by the Newport News Shipbuilding & Dry Dock Co. Each liner was shipped to the job in two pieces and assembled and welded in place. They were anchored by bolts embedded in the concrete. Loop bars were also placed in the concrete to which turnbuckles were anchored for pulling the liners into final shape prior to concreting.

Speed ring erection.—The speed rings were cast in four sections, each section weighing approximately 15 tons, and were shipped to the job unassembled. The speed rings were attached to the draft tube liners after concrete had been poured to elevation 824. Twelve pedestals were poured in which speed ring anchor bolts were set and which served as jack bases. The speed rings were brought to line and grade by means of the anchor bolts and a set of 12 jacks, one placed on each pedestal. These jacks were left in place and were later covered with concrete.

Scroll case assembly.—After the speed rings were in place, the spiral casings were assembled and bolted completely. Erection was started opposite the Y-casting and carried first in an upstream direction to the penstock connection. When this portion had been completely assembled and bolted up, erection was started again at the Y-casting and proceeded in a downstream direction and around the speed ring to the connection with the Y-casting and the previously assembled section. The entire casing was completely assembled and bolted in place before any rivets were driven. The casing was also bolted to the speed ring as the assembly progressed. The Newport News Shipbuilding & Dry Dock Co. furnished the spiral casings and the additional transition section necessary to connect the inlet end of the scroll casing with the penstock.

FIGURE 206.—*Speed ring assembly.*

When the entire spiral casing had been completely bolted, the circumferential joint at the upstream end of the thimble section was riveted. The next joint downstream was then cut in place providing a minimum clearance of 1/8 inch between ends of adjacent plates at this point. Riveting of the 10-foot penstock extension to the welded penstock pipe was the next step. The longitudinal joints in this 10-foot section were also completed at this time. With the exception of the first circumferential joint downstream from the end of the thimble section, the entire spiral casing was then riveted. The unriveted joint was left loose so that any movement occuring during the riveting or final lining up and leveling of the speed rings would not be restrained. This procedure was similar for each unit except that in the west unit the unriveted joint was at the end of the welded penstock pipe. Bulkheads were provided so that space would be left in the concrete to permit riveting of the closure joints at a later date. Riveting of both closure joints was completed during the latter part of January 1936 when both the outside tempera-

ture and the temperature of the exposed portion of pipe were fairly low. The specifications required that these two joints be riveted when the air temperature was 32° F. or below in order to take advantage of the cooling effect of the surrounding air which caused the pipe to contract. This restriction eliminated the possibility of tension after the pipe had been embedded in concrete and the cooling action of the water passing through the pipe took effect.

Both spiral casings contained approximately 39,540 rivets. A total of 1,411, or 3.63 percent, driven rivets were rejected, cut out, and redriven. The best day's driving with seven crews working 6-hour shifts was 2,409 rivets, while the best driving for a 6-hour shift by a single crew was 439 rivets.

Concrete around the scroll cases was poured in 10 lifts varying from 2 to 5 feet in depth. The space under each scroll case adjacent to the bottom speed ring was filled by means of three 12-inch pipes arranged to conduct the concrete into this space which became inaccessible after the concrete reached the bottom of the spiral casing. These pipes were left in the concrete. A series of 2-inch holes in the bottom of the speed ring castings was provided for grouting the bottom of the speed ring. These holes were tapped for 2-inch pipe and were left open to act as air vents during the concreting of the space around the scroll cases. After the concreting was completed, a 2-inch pipe, tapped to fit these holes, was used to fill the remaining space around the casting with grout. Holes were then closed by means of 2-inch pipe plugs which were chipped off flush with the casting. Concrete around the spiral casing utilized a hopper and a series of chutes and elephant trunks.

Three different classes of concrete were used in the pours around the scroll casings. The majority was composed of 3-inch maximum size aggregate using a water-cement ratio of 0.60 by weight and 1.33 barrels of cement per cubic yard. In parts of two or three pours the regular 3-inch reinforced concrete mix was also used. The remainder of the concrete, except for that used in grouting the speed ring, was composed of 1½-inch maximum size aggregate, using water-cement ratios of 0.60 and 0.55 with 1.50 barrels of cement per cubic yard. In grouting the speed ring of the east unit, the mix was composed of ¾-inch maximum size aggregate with water-cement ratio of 0.55 by weight and 1.70 to 1.80 barrels of cement per cubic yard.

Superstructure.

Structural steel.—Approximately 915 tons of powerhouse structural steel were furnished and fabricated by the Virginia Bridge & Iron Co. at a bid price of $62,980. Erection was by the regular job rigger forces under the supervision of the general rigger foreman. All steel was handled to the job from the Coal Creek unloading yard by heavy-duty float trailers. The cableways were used to place the steel in the powerhouse structure.

Accepted practice and the specifications were followed in placing and riveting. A total of 25,930 field rivets was driven, of which about 4 percent had to be cut out and redriven. Erection of the powerhouse steel was started on August 14, 1935, with the setting of

the main column base plates. All the steel was erected and riveted by December 8, 1935.

Concrete.—Forms for the powerhouse superstructure walls presented the most unusual form problem on the entire job. The outside wall surface forms above the water table at elevation 869 were composed of panels 8 feet high and 20 feet long. They were built on a scaffold and moved into place when completed. This scaffold was carried up as the work progressed. Below the water table, the walls were thick enough to allow workmen room to work inside. These forms, as well as all inside wall forms, were built in place. Above the water table the inside wall form was always completed before the outside wall was moved into place. Wales consisted of 6-

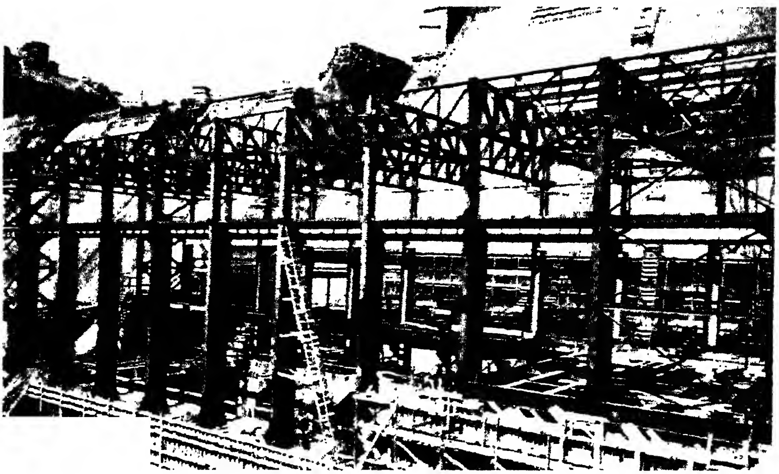


FIGURE 207.—*Structural steel erection.*

by 8-inch timbers on 3-foot centers while studs were 2- by 8-inch timbers on about 15-inch centers. Lagging was placed in two layers: the first layer composed of 2- by 6-inch and 2- by 8-inch pine boards placed diagonally across the studding at about 45°; the second layer on the inside wall surface of the form consisted of ⅞-inch matched tongue-and-groove flooring with the joints horizontal. Considerable care was exercised in scraping and sanding the finished inside wall surface forms to insure that the form marks would be reduced to a minimum in the finished concrete. The second layer of lagging on the outside wall surface form was ⅞-inch matched tongue-and-groove flooring with the contact face unfinished. It was placed in panels 4 feet high and 10 feet long with the joints between boards in one section horizontal and in each of the four adjacent sections normal to the horizontal. At the joints between panels a V-strip was placed to isolate completely each panel from all adjacent panels and to emphasize a large block stone-matched joint effect. Above the water table after the inside wall surface form was completed for

each lift, reinforcing steel was placed prior to moving the outside wall form into place.

Concrete was mixed and transported to the forms by the regular concrete mixing and handling facilities. The 6-cubic-yard, round, controllable dump bucket was used for placing concrete because of the convenience of the controllable dump feature which permitted distributing a batch over a long length of wall form. As a general rule, only 3 cubic yards were handled by the bucket at one time. Concrete was consolidated by means of electric vibrators in the same manner and using the same equipment as for the mass concrete of the dam except that the Mall electric vibrator was used in heavily reinforced sections. The walls were poured in 8-foot lifts.

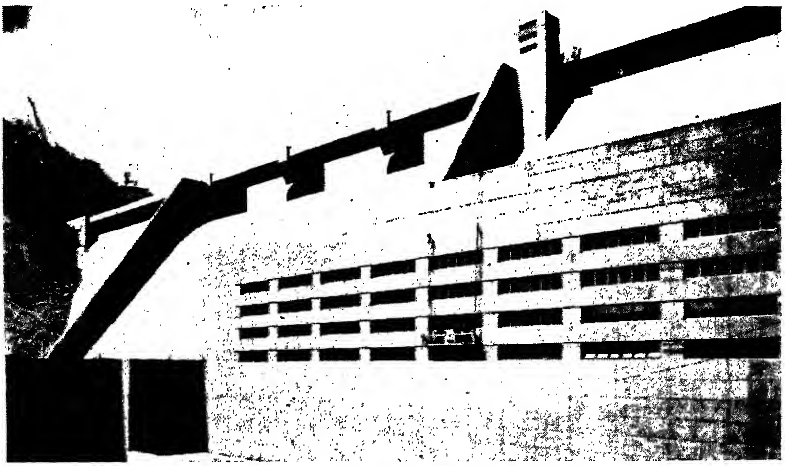


FIGURE 208.—*Exterior powerhouse walls.*

Concrete for the thickest section of the wall is composed of 3-inch maximum size aggregate, using 1.33 barrels of cement per cubic yard and a water-cement ratio of 0.55 by weight. The majority of the concrete in the walls is composed of 1½-inch maximum size aggregate with 1.50 barrels of cement per cubic yard and a water-cement ratio of 0.55 by weight. Extra cement up to about 100 pounds per 3-cubic-yard batch was added at times to improve workability and to aid in placing the heavily reinforced sections. In some very heavily reinforced sections of the walls, the concrete was composed of ¾-inch maximum size aggregate with 1.70 barrels of cement per cubic yard and water-cement ratio of 0.55 by weight.

INSTALLATION OF PERMANENT POWERHOUSE EQUIPMENT

Turbine installation.

Assembly of the turbines was continued after the concrete had been poured around the scroll cases up to and including the generator barrels. When the curb and crown plates were finished in the shop

of the Newport News Shipbuilding & Dry Dock Co., approximately 0.016 inch of stock was left to be removed in the field by grinding, thus assuring a seat for the removable wearing ring, concentric with the remainder of the turbine installation.

Assembly for grinding.—The process of assembling the necessary parts of the turbine and the grinding machine for the grinding process was as follows:

1. The curb plate was placed into approximate final position.
2. Eight of the 24 wicket gates (every third wicket gate) were installed. This included fitting the top and bottom gate stems to their respective bearings in the crown and curb plates. A complete fit was necessary in order that curb and crown plates could be properly lined up with respect to each other prior to grinding.
3. The lower bearing, shaft, and rotating arm of the grinding machine were lowered into place with the driving and grinding mechanisms assembled. Two 10-inch I-beams which rested on the speed ring served to support these grinding machine parts while the remainder of the assembly was completed.
4. The crown plate was then installed.
5. With the crown plate and curb plate both installed in approximate positions, they were lined up properly and doweled to the speed ring.
6. The bearing housing was next installed.
7. With the bearing housing in place, the grinding machine shaft top bearing was installed and the assembly of the grinding machine completed.
8. The two 10-inch I-beams which had previously supported the grinding machine were removed after the machine was completely assembled.

Details of grinder.—The grinder consisted essentially of a vertical shaft, a rotating arm which carried the grinding tool, a driving mechanism, and a support for the weight of the machine.

The vertical shaft was held in a central position by a casting which fit into the conical bore in the bearing housing of the turbine, and was provided with a steady bearing at its lower end to assist in maintaining perfect alignment. The driving mechanism rotated about the vertical shaft on a stationary sleeve fitted with tapered roller bearings, and was driven by a three-horsepower motor through a pair of spur gears, a worm gear, and a pinion gear. The pinion gear meshed with a large gear keyed to the sleeve. The sleeve carried the weight of the revolving parts of the machine and was moved up or down along the vertical shaft by means of a hand-operated screw. This sleeve had four clamping screws at each end, which were used to center the sleeve on the shaft and hold it rigidly during the grinding operation. The grinding wheel and its driving motor were mounted on a saddle, which could be moved in horizontal and vertical directions by means of feed screws. Electric current was delivered to the motors through the brushes and slip rings. The necessary controls and push-button switches were mounted on the machine within easy reach of the operator.

Grinding procedure.—Grinding of the seats for the stationary wearing rings on the curb and crown plates was probably the outstanding feature of the turbine installation. The procedure followed

in grinding the wearing ring seats to final size concentric with the remainder of the turbine was as follows:

1. After the grinding machine was completely assembled, it was plumbed and lined to perfect center. This was done by means of the leveling screws in the castings which fitted into the conical bore of the bearing housing, and by means of the steady bearing at the bottom of the shaft. It was essential that this shaft be perfectly centered and plumbed, because all measurements during the grinding process were made from it.

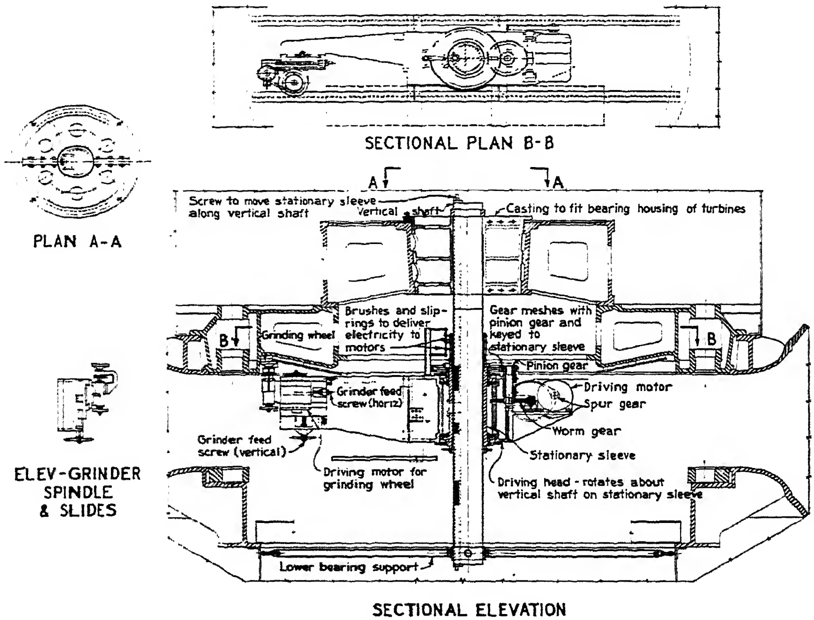


FIGURE 209.—*Details of the special radial grinder.*

2. Seats were next honed down in four or five places to finished neat lines. These spots were approximately ¾-inch square and were placed at the top of the curb plate and bottom of the crown plate. They served as guides to the operators of the grinding machine as to the depth of cut remaining, necessary to bring the seats to the desired neat line.

3. The actual grinding was started on the curb plate at the top of the seat and worked downward, across the plate to the bottom. Cuts were usually about ⅛-inch wide and about 0.001 inch to 0.0015 inch deep. The driving mechanism was arranged to work at 4 speeds, ranging from approximately 1 trip around the periphery per minute to 1 trip in 3 minutes. At the beginning of the grinding procedure, the machine was tried at all speeds but was found to operate more successfully at the slowest speed. From 45 to 50 com-

plete trips around the periphery of the crown or curb plates were usually required to traverse completely the face of the wearing ring seat.

4. After completely traversing the width of a seat, the face was checked for plumb from the shaft of the grinding machine by means of a specially made steel measuring rod. Due to wear on the grinding wheel in traversing the surface of the face, for which no correction was attempted, there always occurred a discrepancy between the edge on which the grinding was started and that on which the grinding was completed. This necessitated some additional grinding in order to make the face of the seat perfectly plumb again. This process was known as "dressing up" the face.

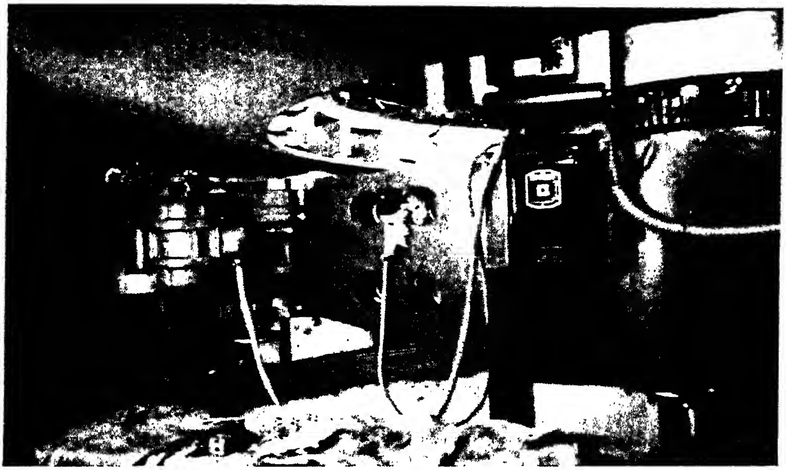


FIGURE 210.—*Grinding operation*

5. Following each operation, the machine was set for another cut and the face of the seat traversed in the opposite direction.

The process of grinding and dressing up was repeated until the seat was ground to the desired neat line. Two daily determinations were made of the status of the grinding, and this information was passed on to the shift machinist foremen for their information as to how much grinding remained to be done. The west unit was ground first, and considerable experimenting was done initially to determine the operating speed, grain size, and grade of grinding wheel that gave best results. It was found that a combination of medium grain carborundum wheel worked best on this steel.[20] The amount of experimenting necessary at the start of the grinding process resulted in 12 working days required to complete the west unit. The east unit was completed in 8 working days.

Wearing rings.—Following the grinding process, the crown plate for each unit was placed bottom side up on the assembly bay floor

[20] Class II, medium-annealed cast steel, Federal Specification QQ-S-681.

and the outer top stationary wearing ring, which had been drilled and countersunk in the contractor's shop, was fitted to the crown plate and used as a template for laying out the assembly holes. This ring was assembled to the crown plate with special double-headed bronze screws and each screw countersunk to take the real head with a groove in the countersink provided alongside the screw head at the surface of the ring for locking the screws. After the screw was set, the false head was cut away and each screw was locked by peening the real head into the groove provided. The head of each screw was then ground flush with the wearing ring surface. The inner top wearing ring, the seat for which was machined in the contractor's shop, was also assembled at the same time and in the same manner as the outer ring, except that all holes had been drilled and tapped in the contractor's shop. This ring was left for field assembly to allow a working clearance for the grinding machine at the crown plate.

The curb plate, which had been assembled to the speed ring at the beginning of the grinding process, was left in place when the grinding was completed. The procedure in assembling the bottom stationary wearing ring to the curb plate was similar to that in assembling the outer top ring to the crown plate.

Completion of assembly.—The remaining steps in the turbine assembly were completed in the following order.

1. The remaining wicket gate stems were fitted to their respective bearings in the crown and curb plates. The gates were removed to permit lowering of the 75-ton runner and shaft into place—temporarily supported on the ledge at the bottom of the speed ring. Assembly of the shaft to the runner had been previously completed on a timber crib built on the 847 floor of the powerhouse, downstream and midway between the two units.

2. All gates were assembled to the curb plate, the crown plate was placed, and the installation of the wicket gates completed with the assembly of the individual gate bushings and wearing plates.

3. The second step was followed by the assembly of the packing box housing and bearing housing to the crown plate; installation of wicket gate levers and links; and the installation of servomotors and connecting rods.

4. The guide was next assembled and fitted so that the turbine shaft and runner could be checked for centering and plumb prior to placing the top part of the shaft which was a part of the generator contract. This check was necessary since after the generator assembly was started, no further alignment of the turbine shaft was contemplated until final alignment and centering after the generator rotor was in place.

5. After the completion of the above operation, the guide bearing was removed, the top half of the shaft placed, and the connection between the top and bottom sections made. Assembly of the generator was then continued until the entire weight of the runner and shaft was taken by the Kingsbury bearing. At this stage of the generator assembly, the combined shaft was tested for plumb. This was done by measurements from four heavy plumb bobs suspended on piano wires from the lower bracket of the generator. Adjustment of the thrust bearing bolts was made to bring the as-

sembly to perfect plumb. The runner and shaft were rotated through 360° and tested for plumb from the plumb bob suspension wires at each quarter point. The combined shafts were also checked for trueness over the entire length by mounting indicators in pairs at several elevations and rotating the runner and shaft. Prior to this test, the gland packing was installed so that pendulum motion of the shaft and runner would be avoided during rotation. This check revealed that the shaft assembly in each unit was perfectly true within the tolerances accepted in installations of this kind.

Erection of the generator was continued after this check, with the installation of the completely assembled rotor. With the rotor in place, the final check on vertical alignment of the shaft was made to determine whether or not any unequal deflection of the lower bracket had occurred due to the addition of the rotor load. The same procedure was followed and the same equipment used as in making the original test for plumb.

6. Final installation of the guide bearing was completed after the final check on vertical alignment of the shaft was made. In the west unit, the guide bearing was assembled and checked for centering and alignment and found to be correct. When assembly of the guide bearing was attempted in the east unit, it was discovered that misalignment of turbine parts had occurred. A check of the completely assembled west unit revealed objectionable misalignment that had occurred after final check.

7. The remainder of the installation entailed the assembly of the miscellaneous small parts which included the stuffing box ring, guide ring, bearing cover, ring seating, indicating devices, etc. Standard mechanical practice was followed in fitting the parts together, and specifications were adhered to as rigidly as possible in regard to tolerances and clearances.

Assembly personnel.—The contractor furnished three erectors who supervised the work of installing the turbine. The first erector was sent to the job just prior to the installation of the draft tube pit liner, and served as contractor's erection superintendent on this work. Prior to the installation of the speed rings and spiral casings, a second man was sent to serve as contractor's erection superintendent. He was assisted by the erector who had supervised the installation of the draft tube liner. After the spiral casings were completed, a third man was sent to serve as turbine erection superintendent. He relieved the erector who had supervised the speed rings and spiral casings, but retained the same man as assistant who had been assistant on the spiral casings assembly.

For the Authority, the turbine installation was under the general supervision of the master mechanic. Grinding of wearing ring seats was carried on during three 8-hour shifts, requiring one machinist foreman and crew on each shift. Another machinist foreman and crew on each shift handled the remainder of the erection which was carried on during two 8-hour shifts.

Turbine misalignment.

The first indication that some outside force had acted to cause misalignment of the turbine assembly came during the installation of the east unit when the turbine erector was ready to make the final

installation of the guide bearing. Considerable difficulty was being encountered by the generator erector in correcting the vertical alignment of the shaft after placing the rotor. This misalignment was attributed to the unequal deflection of the generator lower bracket by the addition of the rotor load. Acceptable alignment was finally obtained, but in order to center the runner again it was necessary to shift the guide bearing. About 6 weeks prior to this time, a preliminary installation of the guide bearing had shown the runner centered and the shaft plumb. This discovery in the east unit led to an investigation of the condition of the alignment in the west unit, which had been completed and coupled to its generator about 2 months before. The investigation revealed that a similar but greater misalignment of turbine parts had occurred.

Condition of assembly.—A check to ascertain the magnitude of the misalignment indicated that the top of the shaft of each unit had moved upstream and eastward on a line about 45° with the axis of the dam. In the west unit the magnitude of the major deformations was greater—being in most cases about twice as great as in the east unit. Besides the tilting of the shafts the guide bearings and speed rings were out of level and the latter had become slightly elliptical with the minor axis parallel to the line of movement of the turbine shafts. The wicket gates were found to be out of square with the speed ring and caused binding in the gate bearings. Runner clearances had not changed appreciably at the crown plate of either unit, probably due to the relatively stiff crown plate which withstood the external forces tending to deform it. As an indication of the magnitude of the misalignment, the shaft of the west unit was out of plumb 0.059 inch in 13 feet and runner clearances at the curb plate showed a difference of about 0.054 inch between the length of the major and minor axes of the speed ring.

Cause of distortion.—No definite cause of the distortion of the turbine installation was ever determined. It was thought at the time of the discovery that some structural deflection or foundation deformation resulting indirectly from water load on the dam might have caused it. Several cracks in the powerhouse substructure and superstructure concrete which had previously been noted were observed and studied in an attempt to substantiate this theory, but no definite conclusions were reached. The position and direction of the cracking, however, seemed to indicate in a general way that the theory of structural deformation caused by water load was essentially correct.

Corrective measures.—After considerable study, an agreement among representatives of the Newport News Shipbuilding & Dry Dock Co., Westinghouse Electric & Manufacturing Co., and the Tennessee Valley Authority was reached as to the procedure to be followed in realigning the turbine assembly. The following procedure was adopted for the west unit:

1. Without moving the thrust bearing and top guide bearing, the runner was placed so that the following clearances were obtained between the runner and crown plate wearing ring:

	Inch
Downstream (tailrace position)	0.037
Upstream (opposite tailrace)	.029
East and west sides—equal at	.025

2. The lower or turbine guide bearing was then adjusted, and clearances, shown in figure 211 (a), between guide bearing and shaft were obtained.

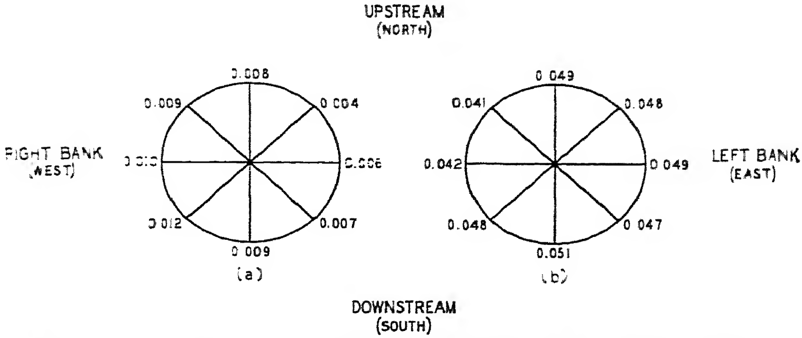


FIGURE 211.—*Clearances in inches between guide bearing and shaft—west unit.*

3. Without moving the shaft, the thrust bearing was adjusted to distribute properly the load on the bearing.

4. The curb plate was shifted until approximately equal runner clearances were obtained all around as indicated in figure 211 (b).

No attempt was made to plumb the shaft, and a check after all the above adjustments were made showed the shaft to be about 0.051 inch out of plumb in 13 feet with the top upstream and east from the bottom. The thrust bearing was adjusted to maintain the shaft in this position.

In the east unit, it was decided to continue the normal procedure of alignment. The distortion was much smaller in this unit than in the west unit, so it was possible to make the necessary adjustments to bring the parts into almost perfect alignment. The shaft was approximately 0.010 inch out of plumb in 13 feet with the top downstream from and west of the bottom. Figure 212 (a) indicates the clearances obtained between the top of the guide bearing and shaft. Final clearances between the runner and the stationary wearing ring at the crown plate varied from 0.040 to 0.033 inch and are shown in figure 212 (b). These clearances showed no appreciable change from those which existed at the time the distortion was discovered. No attempt was made to adjust the curb plate to alter the clearances between the runner and the stationary wearing at the curb plate. The clearances were considered ample as shown in figure 212 (c) and, although not absolutely uniform, were considered acceptable under the circumstances.

Conclusions.—Several of the conclusions reached regarding the turbine misalignment are as follows:

1. The same outside force which acted to cause distortion of one unit also caused distortion in the other. This is obvious from the fact that the same manner of misalignment occurred in both units.

2. The difference in magnitude of misalignment in the two units was largely due to the fact that the force acted after the west unit was completed, thus causing the west unit to reflect the full effect.

whereas the east unit was in process of assembly and only reflected a part of the effect of the force.

3. Apparently the effect was of a permanent nature, since measurements on the level of the speed ring in the crown plate landing have indicated no change in magnitude of misalignment over a period of 1 year.

4. The careful grinding of the wearing ring seats to insure that the wearing ring would be concentric with the remainder of the installation was of little value under the circumstances.

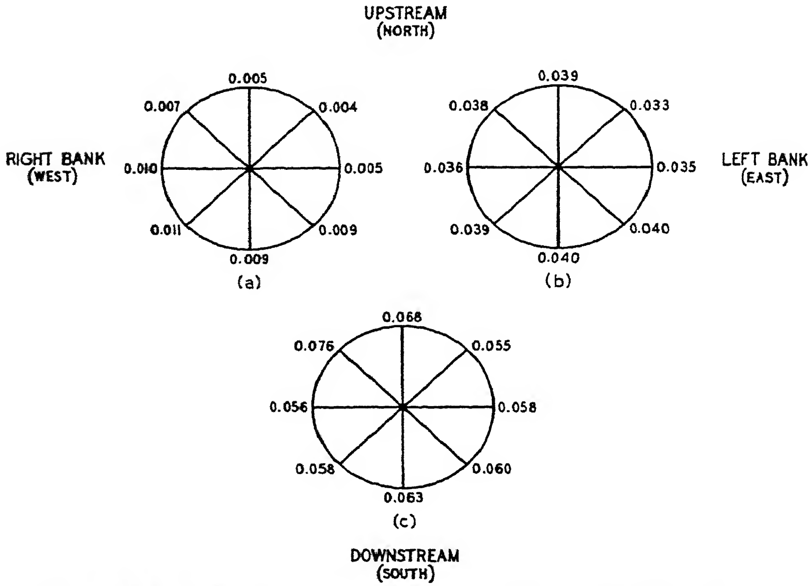


FIGURE 212.—*Clearances in inches between guide bearing and shaft—east unit.*

5. Although not desirable, the remaining misalignment in the two units is minor. This conclusion seems valid in view of intermittent operations of both units at varying loads and heads over a period of approximately 1 year. Further developments after a longer period of operation may change this viewpoint.

6. Units at other plants have been run for several years with greater misalignment than occurred at Norris with no injuries to the bearing or any other part of the assembly.

Operating Noises and Vibrations.

The west unit was first put into operation on July 28, 1936, under a head of 176 feet. Except for a slight whistle at very low loads, which seemed to come from the generator, the unit was quiet at all loads and gate openings and the slight whistling noise disappeared after a few hours of operation. Operation was almost continuous

after the first few days at very nearly full loads, until about September 30, 1936. Noise and vibration were first noticed about September 20, 1936. After a shutdown on September 26, the vibration was much more serious and the noise more noticeable. The head at this time was about 160 feet, and the noise and vibration were greatest at gate openings between 50 and 75 percent.

In the east unit similar vibrations developed when the unit was under a load of 36,000 kilowatts which required a gate opening of 65 percent at 160-foot head.

Corrective measures.—After observing the two units in operation over a period of several days, attempts were made to learn the cause of the noise and vibration. During the period of observation, an oscillograph was obtained and the period of vibration was determined to range between 90 and 100 cycles per second. When the machine was shut down and the runner unwatered, another test was conducted to determine the natural vibration period of the runner vanes. This consisted of tuning a trombone to the tone obtained by striking a runner vane with a hammer. The natural period of vibration of the runner vane was found to be about $1/96$ of a second, which corresponded remarkably well with the frequency of vibration obtained by means of the oscillograph. This somewhat substantiated the theory that the noise and vibrations were the result of a hydraulic phenomenon that caused vibration of the runner vane, which was subsequently imparted to other parts of the turbine.

In an attempt to eliminate the noise and vibration, a series of tests was run on the west unit using wood blocks and 1½-inch round steel bars as stiffeners between the runner vanes. The tests conducted and the results obtained may be outlined as follows:

1. Wooden blocks as struts were placed between the runner vanes. A test was made, and it was discovered that the range of gate opening at which the noise and vibration had occurred had changed from the original range of from 52 to 72 percent to a range of from 63 to 72 percent. During this test run, it became necessary to drop instantaneously the load of 30,000 kilowatts which the generator was carrying, resulting in a momentary runaway, and causing the loss of some of the wooden blocks.

2. The loss of some of the blocks, as outlined above, made it advisable to make another effort along similar lines. Therefore, the blocks were replaced and, in addition, the bottom half of the runner tip was removed and the small openings in the upper half of the runner tip were closed by the application of a plate on the inside and outside of the runner tip. A test run was made, and, although the noise seemed to be lessened over the full range of gate opening, it was found that the no-load gate requirements were raised from a normal of 4 percent to 10 percent. It was also found that the full load capacity of the generator was reduced by between 2,000 and 3,000 kilowatts. Vacuum conditions were also affected to the degree that no vacuum existed up to a load of approximately 25,000 kilowatts. In view of the above, it was concluded that no appreciably better results were obtained from this test.

3. The wooden blocks were again checked and the bottom half of the runner tip left off, using in place a flat steel plate attached to the bottom of the upper half of the runner tip. The plates over the

holes in the upper half of the runner tip were removed. A test run was made, with the result that vibration and noise were eliminated over the full range of gate opening, with the exception that between 65 and 68 percent gate openings, there was a very slight vibration and almost imperceptible extraneous sounds.

This test run appeared to indicate conclusively that the vibration and noise conditions were due to the vibration of the runner casting itself, and that by bracing between the vanes the vibration could be completely eliminated. It was felt that the vibration existing over the 3 percent gate opening above mentioned was doubtless due to the fairly large area of obstruction created by the size of the wooden

FIGURE 213.—*Struts welded to turbine runners to eliminate vibration.*

blocks. Therefore, it was decided to reduce this area by tack welding a 1¼-inch round steel bar between the vanes at approximately the same location at which the wooden blocks had been placed.

4. The blocks were removed and replaced by 1¼-inch round steel bars between the vanes and tack welded on a level with the curb plate and 26 inches in from the edge of the runner. The bottom half of the runner tip remained removed, and the flat steel plate attached to the bottom of the upper of the runner tip remained in place, as in the previous test. The holes in the upper half of the runner tip remained open.

A test run over the full range of gate openings from 0 to 100 percent and over the full range of generator loading was made. This run demonstrated conclusively that no vibration existed at any point in the entire range, and that the unusual noise condition was completely eliminated. The no-load gate requirement returned to the previously indicated normal position of 4 percent gate opening and the full load capacity of the machine returned to the previously es-

tablished normal rating of 48,000 kilowatts. This was obtained at the total head of 159.2 feet.

As a result of the above experiment and test runs, it appeared conclusively that the noise and vibration originated in the vibration of the runner vanes. The experiments with the struts seemed also to indicate that to eliminate the vibration effectively, it would be necessary to tie the vanes together, thus reducing the unsupported length and changing the natural vibration period.

An agreement was reached whereby the Newport News Shipbuilding & Dry Dock Co. furnished and installed the necessary struts. The danger of increased cavitation and pitting was considered, and it was decided to use struts elliptical or "fish-shaped" in cross section instead of round bars in order to reduce this possibility. Struts were installed in both runners, with the result that practically all vibration and noise have been eliminated.

Governors.

The installation of the governor on each unit involved no special difficulties. This equipment was installed completely, including all piping, electrical connections, and wiring, by the regular electrical and mechanical forces of the Authority. After the installation was complete, the contractor's engineers made all necessary adjustments and placed the governors in operating condition.

Generator erection.

Erection of the generators was under the supervision of an erection superintendent from the Westinghouse Electric & Manufacturing Co., assisted by one assistant superintendent, one foreman, one mechanic, five winders, eight helpers, and one timekeeper and office assistant. For the Authority, erection was in general charge of the general electrical foreman. The only additional job personnel actually engaged in the erection were the riggers who handled the unloading and transporting of the parts to the job and the powerhouse crane operator who operated the crane during the erection. Most of the work was of a general assembly nature and was completed with little difficulty.

Rotor assembly.—The rotors were shipped to the job completely dismantled and the entire assembly made on the job. In order to facilitate handling by railway freight from the factory to the Coal Creek unloading yard, 4 of the 10 spider arms were cast about 4 feet shorter than the required length. Four extensions to match the short spider arms were cast and furnished for connecting by heavy bolts to bring the arms to proper length. Except for the 4 extensions, the spider for each rotor was cast of steel in 1 piece with a solid hub.

Standard electrical practice and specifications were adhered to throughout the rotor assembly which was carried out in the following steps:

1. The spider was placed on the stub shaft which rested on a special base built into the erection bay floor.
2. Spider arm extensions on four short arms were assembled and bolted.

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3. Brake ring was fitted and removed.

4. Lower end plate was assembled, laminations stacked, and the upper end plate assembled, completing the assembly of the laminations.

5. Laminations were expanded by heating and driving wedges between the end of the spider arms and lamination ring plates.

6. Brake ring was assembled.

7. Field coils were hung.

8. Field coil connectors, damper coil ring, and ring connectors were placed.

9. Field coil resistance was checked.

Stator assembly.—Each stator was shipped to the job in four equal segments which were assembled on the generator barrel and the frames welded together. The field erection included six steps:

1. The four segments of the stator were matched and the frames welded.

2. Coil slots were insulated and the remaining 16 coils on each segment were hung and pegged.

3. Connecting rings were placed and insulated and the coils connected.

4. Each field-erected section of coils was tested at 32,000 volts for insulation breaks.

5. Field-erected coils were dried out by heating with a welding machine at 10 volts and 600 amperes direct current.

6. Resistance of each phase to ground was checked.

Mechanical assembly.—Except for miscellaneous instrument installations, the remainder of the field assembly was of a mechanical nature. After the stators and rotors were completed, the associated parts were placed in the pit, and the proper adjustments made to bring the unit into operating condition. The field procedure to complete the generator assembly and make connections to the turbine was as follows:

1. The lower bracket which had been shipped to the job with the arms dismantled from the hub was assembled and placed in the pit on the sole plates.

2. The thrust bearing runner was attached to the upper part of the shaft and the assembly placed in the pit with the shaft resting on the turbine shaft (see fig. 58).

3. The two halves of the thrust bearing stator were placed around the shaft and jacked into position for bolting to the lower bracket.

4. By means of the thrust bearing screw jacks, the shaft was lifted to the proper elevation and line.

5. The stator was then lined up with the shaft and the two shafts coupled together with four bolts to lift the runner and shaft into final position before completing the shaft connection.

6. Both shafts were checked for trueness and were centered in the turbine guide bearing.

7. The shaft was checked for plumb and centering before placing the rotor.

This completed the assembly with the exception of placing the upper bracket, exciters, coolers, and ventilating housing.

Field tests.—The generator contract specified that certain tests were to be made after the two generators were completely erected. These tests are described in chapter 3 and the results are given in appendix F.

Penstock trashracks.

A decision was reached, before concreting was started in the blocks of the dam in which the trashracks are located, to carry the concrete for the dam up ahead of that for the trashracks. This decision was made because it was necessary to carry the concrete to the necessary height as rapidly as possible for installation of the penstocks so that the penstock contractor would not be delayed. Also, the large amount of formwork and reinforcing steel in the trashracks would have slowed up pouring in blocks 34 and 35 if the trashrack structures had been poured monolithically with the dam. Also, it was easier to hold the trashrack forms to line and grade after the main structure was completed. It was necessary to build an auxiliary cofferdam arm tying the upper arm of cofferdam No. 1 to block 38 in order to unwater the area where the trashracks were to be built. The necessary excavation for the trashrack structures was also completed after the unwatering of the auxiliary cofferdam.

Forms.—Forms for the trashrack structures were fabricated in the carpenter shop. Each beam form was built in one piece and cut into eight sections at the points which represented the center line of columns. Column forms were fabricated in approximately 12-foot lengths. The entire form was assembled in the field.

Lagging for prefabricated forms consisted of 1⅝-inch S4S No. 2 matched pine. Bracing for form erection was based on the judgment of the carpenter foreman in charge. Special care was exercised in building the beam and column forms to make certain that the finished surface of the lagging conformed as nearly as possible to the neat lines of the structure. This was accomplished by cutting the lagging to very narrow widths in order to approximate the neat lines of very short radius curves. No attempt, however, was made to finish the surface of the lagging to true curved lines; and as a result, these became a series of very short chords in the finished form and structure.

The general features of form erection and concreting were carried out in the following order:

1. After completion of the base of a trashrack structure, the first set of column forms was placed to line and grade and the top of each form cut in place to fit the first beam form.

2. Beam forms were then placed to line and grade and the reinforcing steel placed for the columns but not for the beams.

3. The columns were poured to a point coincident roughly with the lowest point of the beam and column intersection.

4. Beam reinforcing and anchor bolts for the trashrack steel were then placed and the beam concrete was poured. A space at the top of each beam form about 18 inches wide was left unlagged, through which the beam concrete was placed.

Following these steps, another set of column forms was erected and the process outlined above repeated for each succeeding set of columns and beams. Scaffolding was carried up inside the trash-

rack. Platforms from which the forms, reinforcing steel, and concrete were placed were provided at approximately the elevation of the center line of each set of beams. Reinforcing steel for the columns was assembled before placing on these platforms.

Concrete.—A concrete containing 1½-inch maximum size aggregate with 1.70 barrels of cement per cubic yard and water-cement ratio of 0.50 by weight was used in the trashrack structures. It was handled by cableway to the forms in the round controllable dump bucket and consolidated, using both electric and air vibrators. Since this concrete was poured during extremely cold weather of the winter of 1935–36, it was necessary to provide a heating plant in order to protect the concrete from freezing in the thin sections composing the structure. The heating plant consisted of a vertical steam boiler located on the east bank of the river upstream from the dam, and a system of perforated pipes arranged around the trashrack structures. A swinging walkway was provided to give access to the structures from the boiler, and this walkway also supported the steam line of the perforated pipe system.

Metalwork.—The metalwork, which included the guides, anchor bolts, and racks for the trashrack structures, was furnished by A. J. O'Leary & Sons Co. The erection and installation was done by the Authority's forces. Prior to installation, the racks were hot-dipped in a bath of "Ovarco" hot-dip asphalt in a specially built steel tank at the railroad siding at Coal Creek.

Progress.—Construction of the trashrack structures was started in mid-December 1934 when excavation for the base slabs was begun. This excavation required about 1 week. The first concrete pour was made in February 1935 and concreting was completed for both units by the end of August 1935. The structures were completed by mid-September with the installation of the guide beams and steel trashrack bars.

Penstock transition section.

The upstream section of each penstock, beginning at the upstream face of the dam and extending downstream to the steel pipe, includes a transition section from a rectangular opening 16 feet 6 inches by 28 feet 6 inches at the upstream end to a circular opening 20 feet in diameter at the beginning of the steel lining. Forms for the transition section, except the ribs, were built in place. Ribs were built in the carpenter shop in two pieces so that the lower half could be erected and lagged and concrete poured around the lower half of the penstock. The top half was then erected and lagged before placing the remaining concrete around the top of the penstock. Lagging consisted of one layer of tongue-and-groove flooring. Ribs were closely spaced so that thin material could be used since it was necessary to bend the lagging to conform to the curved lines of the transition. The inside surface of the forms was sanded to remove all rough edges in order to produce as smooth a concrete surface as possible.

Gate grooves.—Anchor bolts were provided and suitable recesses were made in the concrete adjacent to and just upstream from the upstream end of the transition section to accommodate the installation of gate seats, guides, and seals necessary for the proper functioning

of the gates. The tractor gate slots and the bulkhead gate guides were anchored in two concrete columns which are monolithic with the dam up to the top of the trashrack structure proper. Above the trashrack structure the gate slot was formed by the gate well walls. The bulkhead gate guides extend only 10 feet above the top of the trashrack structure.

The original plans called for a structural frame to support the tractor gate seats and guides during erection and concreting. The steel template idea was abandoned and in its stead the seats and guides were held rigidly by means of the anchor bolts while being aligned and grouted into place. Suitable grooves were left in the mass concrete to receive the cast steel frames. The anchor bolts which served as tension and jacking bolts were accurately set in the

FIGURE 214.—*Tractor gate erection.*

mass concrete to hold the seats and guides and to aid in final alignment just prior to grouting. The results vindicated this procedure as the seats are true to position within 0.060 inch in their height of about 28 feet. A saving of approximately $7,500 resulted.

Concrete.—Because the transition section and gate box columns were heavily reinforced, and it was necessary to use a concrete mix having a maximum size aggregate of 1½ inches, concrete having a very high strength was used. It contained 1.70 barrels of cement per cubic yard and had a water-cement ratio of 0.50 by weight.

Tractor gates.

The delivery date on the penstock gates was considerably later than the scheduled date for final closure of the dam on account of the delay in deciding which type of gates should be used. In order to provide for this contingency, a set of timber stop logs to extend to elevation 880 was provided and installed on the upstream side of each tractor gate area in the bulkhead guide grooves. It was contemplated that the impounded waters would possibly reach an elevation higher than 880; and, to prevent flow of water into the powerhouse area through

the penstock in case the stop logs were overtopped, a bulkhead was built in each penstock. The stop logs and bulkheads were arranged so that the gate area would be accessible to workmen for installation of the guides, tracks, seats, and seals, and also for checking operation of the gates before putting them into service. After the gates were erected, the temporary stop logs were removed and, with the gate in place, the bulkheads in the penstocks were dismantled and the material removed through the manhole in the scroll cases.

Gate assembly.—The gates were shipped by rail to the Coal Creek unloading yard. Each gate was composed of 4,507 principal parts, but they were shipped to the job in as few pieces as practical to reduce the amount of field assembly work. The handling capacity of a single cableway was a major consideration in determining the maximum size pieces that could be assembled for shipment. The largest single piece to be handled, as partly assembled in the shop, was the lower leaf section, which weighed approximately 41,000 pounds. Each gate was received on the job in 49 pieces, exclusive of 32 trunnion shaft shims and the additional nuts, bolts, and screws necessary for assembly.

The gate for the west penstock was assembled on top of the west intake trashrack structure. The two resting beams, which were later installed at the top of the gate well on which the gate rests when in a raised position, were placed on top of the trashrack and utilized as a temporary assembly platform. Gate assembly proceeded in the following manner:

1. Two end plate stiffeners were bolted to the lower gate leaf section which had been placed on the assembly platform. They provided guides which facilitated setting the center and top gate leaves which were then placed on top of the lower gate leaf and bolted to the end plate stiffeners.

2. The upper and lower roller brackets to which the carriage shoes had been assembled were fastened to the gate leaves. By means of the roller brackets, the gate leaves were lined up with the roller bracket assembly bolts tightened to hold the leaves in line.

3. The tie bar and splice plates were assembled to the lower gate leaves, taking special care to keep the gate leaves in alignment. All bolts in the entire assembly up to this point were tightened.

4. The cross-head assembly was placed and blocked to the approximate position which it would assume with the gate in the raised position.

5. All wedge roller assemblies were hung in place, the adjusting blocks bolted to the upper roller brackets and the chain anchor temporarily wired to the quadruple roller chains.

6. The roller carriage assemblies, without the rollers, were hung in place on the cross-head trunnion shaft.

7. With the roller carriages in place, the wedge roller assemblies were attached to the roller carriages by means of the chain anchor which had been wired previously to the quadruple roller chains to hold the wedge roller assembly in temporary position.

8. Carriage roller assemblies were then placed on the roller carriages, and the ends of the roller assemblies connected to make the endless carriage.

9. All roller carriage guides were assembled and bolted.

10. The vertical gate seats to the gate leaves were assembled and the assembly screws tightened before the upper and lower roller guards were assembled to the cross-head housing and roller carriages, respectively.

After the hoists were in place, the top cover plates were removed from the cross-head assembly to facilitate lacing the hoisting cable around the walking beam sheaves.

The gate which closes the east penstock was assembled on top of the gate well of the intake structure for this penstock. A temporary assembly platform similar to the one used for the west penstock gate was utilized and the same procedure was followed on both gates except that the carriage shoes which were shop assembled to the roller brackets for the west gate were assembled in the field for the east gate.

Excellent progress was made in tractor gate erection. The first shipment of the west gate was received on January 21, 1936, and assembly of the complete unit finished by February 6. Assembly of the east gate was completed between February 13 and 25, 1936.

Tractor gate hoists.

Each tractor gate hoist was shipped completely assembled except for the hoisting ropes, foundation bolts, selsyn transmitter, and some miscellaneous small items. The first hoist was partly dismantled as it was unloaded from the car and loaded onto the float trailer for transfer to the job. To facilitate handling of the frame and motors, the drums and ring gear assembly, with the drum bearings attached, were removed from the hoist frame. The second hoist, completely assembled, was loaded onto the trailer and was handled by two cableways by use of a lifting beam to the top of the dam. It was then dismantled because the available space for placing the hoist was so restricted it could not be placed when completely assembled.

On January 15, 1936, the first gate hoist was received and erection was started over the west intake on January 18. Erection was completed on February 6. Erection of the second unit was started on February 19 and was ready for operation by February 25. Final adjustment of the first unit was completed February 17 and of the second unit on March 1, 1936.

RESERVOIR CONTROL

Spillway apron.

Apron concrete was poured in blocks generally 28 feet wide and 55 feet long with alternate block lines corresponding with the block lines of the gravity section of the dam. However, the downstream blocks which were 50 feet long, and the east row of blocks which were poured monolithic with the base of the training wall deviated from the general plan. Forms for apron concrete were built in place with 2-inch lagging and 2- by 10-inch studs backed by 6- by 8-inch and 8- by 8-inch wales.

The apron was made of spillway face concrete containing 1.20 barrels of cement per cubic yard, water-cement ratio of 0.56 by weight, and 6-inch maximum size aggregate. In the sill it was necessary at times to use concrete containing 1½-inch maximum size aggregate with 1.50

barrels of cement per cubic yard and a water-cement ratio of 0.55 by weight in order to facilitate finishing. Finishing was done by screeding and wooden troweling of all rough spots.

Gravity training wall.

Five blocks made up the gravity training wall; three were 55 feet long, one was 50 feet long, and the fifth, which connected the gravity wall with the west bank, was approximately 40 feet in length. Concrete was placed generally in 5-foot lifts, with the tops of the lifts slightly sloping to facilitate cleaning. At the bottom of the wall, due to foundation irregularities, lifts were varied in thickness. Forms for this concrete were all built in place, using 2-inch lagging with 2- by 10-inch studs placed vertically, and 6- by 8-inch wales.

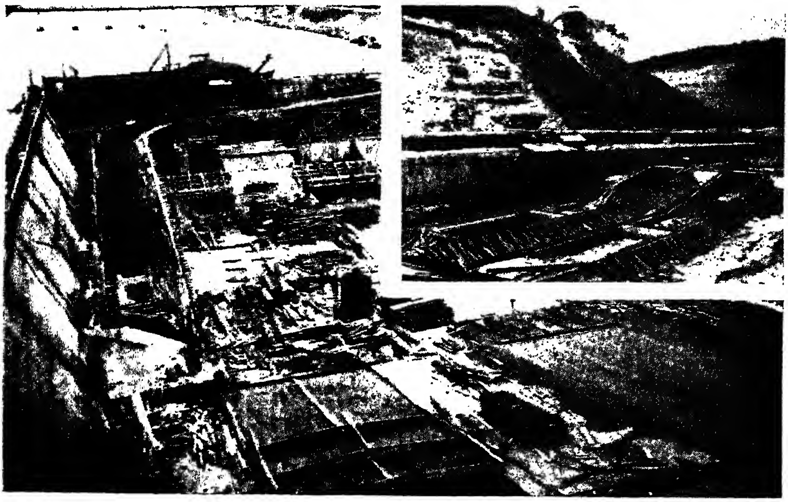

FIGURE 215.—*Apron concreting operations*

On the extreme sides and top, spillway face concrete was used containing 1.20 barrels of cement per cubic yard, with 6-inch maximum size aggregate and a water-cement ratio of 0.56 by weight. The remainder of the wall was face concrete containing 1.10 barrels of cement per cubic yard, 6-inch maximum size aggregate, and a water-cement ratio of 0.58 by weight. Some reinforced mixed concrete with 1.50 barrels of cement per cubic yard, 1½-inch maximum size aggregate, and a water-cement ratio of 0.55 by weight was used at the top to facilitate finishing. Finishing was done in the same manner as for apron concrete.

Cantilever training wall.

Three blocks, 38 feet, 55 feet, and 50 feet long, respectively, made up the cantilever training wall. Forms were similar in design to those used on the gravity wall. Lifts were 9 feet 6 inches deep although

the base of the wall was poured in lifts varying in thickness depending on the nature of the foundation—the thickest lift being approximately 12 feet deep. A reinforced mixed concrete containing 1.50 barrels of cement per cubic yard, 1½-inch maximum size aggregate, and a water-cement ratio of 0.55 by weight was used throughout.

Sloping training walls.

The two spillway training walls, which extend up the face of the dam, are identical, being 5 feet thick at the outer extremity and 8 feet thick at the base or at the point where the walls are coincident with the downstream face of the spillway. Both were poured monolithic with blocks of the dam. Forms were built in place and carried up with the forms for these blocks. They presented an unusual forming problem since the slope on each side of the walls is perpendicular to the sloping face of the dam and the depth of the wall increases from top to bottom, resulting in a slightly warped surface. In order to produce the desired neat line, the forms were built with 2-inch lagging placed horizontally, 2- by 8-inch studs placed perpendicular to the top slope of the training wall, and 6- by 8-inch wales placed perpendicular to the studs. All pours were made in 5-foot lifts. The reinforced concrete mix containing 1.50 barrels of cement per cubic yard was used with 1½-inch maximum size aggregate and a water-cement ratio of 0.55 by weight.

Drum gate erection.

The 3 drum gates were fabricated in the shops of the Virginia Bridge & Iron Co. Intermediate skin plates were fabricated in sections 84 inches wide, each section being riveted to 3 girders, 2 of which were intermediate girders and the third a common girder for joining 2 adjacent sections of skin plate. There were 36 of these completely fabricated intermediate skin plate sections for each gate—12 each for the upstream, downstream, and bottom sections. End skin plates were fabricated in sections 92½ inches long and riveted to the girder common to the end skin plate and the adjacent intermediate skin plate section. Thus, there were 6 end skin plate sections for each gate, consisting of 1 right and 1 left end set for the upstream, downstream, and bottom sections. The end skin plate sections are reinforced with ⅜-inch plates 44 inches wide, which extend from the first standard girder to and under the end bulkhead assembly. These plates were included in the end skin plate assemblies as fabricated in the shop. Both end assemblies for each gate were cut and punched in the shop, but the only other shop fabrication on the end assemblies consisted of riveting the gusset plates to the main framing ship channels and to some of the angles and cross-bracing girders. The remaining parts of each gate which included the seals and seats were fabricated in the contractor's shops and assembled in the field.

Field erection.—Shop-fabricated pieces for the drum gates were shipped by rail to Coal Creek and there loaded on rented trucks for transfer to the dam. A steel stiff-leg derrick on top of block 47 was used by the contractor to unload the trucks. Material was placed on top of the dam and was handled into place for erection by cableway.

Gate erection was handled entirely by the contractor except that the anchor bolts for the hinge anchor and gate seat castings were placed by the Authority. The gates were assembled in the raised position supported on heavy scaffolding over the gate wells. Normal procedure for assembly was as follows:

1. Gate seat castings and hinge anchor castings were set to line and grade, and grouted.
2. Heavy scaffolding was built in the gate well to support the gate in the raised position during assembly.
3. Bottom, intermediate, and end skin plate sections were placed on scaffolding, gate hinges were assembled to hinge anchors and adjacent skin plate sections bolted together at an unriveted joint of a common girder.

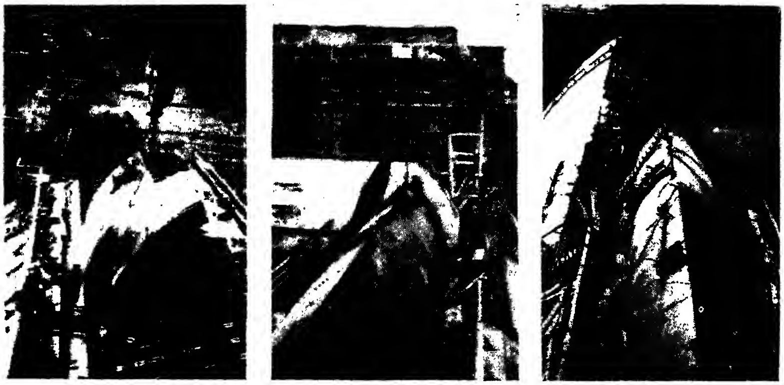

FIGURE 216.—*Drum gate erection.*

4. Intermediate upstream and downstream skin plate sections were placed and bolted.
5. End assembly was placed and bolted.
6. End upstream and downstream skin plate sections were placed and bolted.

Each gate was completely assembled except for minor details before riveting was started. Approximately 10,400 field rivets were required in the assembly of each gate, of which about 3½ percent were rejected, cut out, and redriven. Several difficulties arose during the driving of the rivets which had not been anticipated by the detailer or by the contractor during fabrication. In all cases a satisfactory agreement was reached between the Authority and the contractor and the contingency was corrected. The following is an outline of the corrective measures applied:

Gate No. 1 (east gate)

1. It was impossible to drive 14 rivets specified for each of the ⅜-inch web plates at the ends of the gate because of insufficient room. Machine bolts were substituted and the nuts were welded to the bolts and plates.

2. It was impossible to drive two downstream rivets in each of the connections between the upstream and downstream skin plate girders. Machine bolts were fitted and the nuts welded to the bolts.

3. It was impossible to drive rivets for a connecting angle between two bracing girders at each end of the gate because of insufficient room. Machine bolts were substituted and the nuts welded to the bolts.

4. The holes for the connecting rivets for attaching the radial bracing to the ends of the gate were drilled in the field. Only about half of the rivets could be driven and the remaining holes were welded shut and the brackets welded to the plates as a substitute for the rivets omitted.

Gate No. 2 (middle gate)

1. The situation which arose on gate No. 1 in regard to the ⅜-inch web plates at each end of the gate was identical on the other gates except that on gates Nos. 2 and 3 the plates were riveted to the holding angles and 2- by ⅜-inch bars were tapped for countersunk tap bolts and welded on to the angles for attaching the ½-inch end seat plates.

2. It was necessary to fillet weld at one intermediate girder where the girder and downstream skin plate were not properly drawn up before riveting.

3. Same as item 2 gate No. 1.

4. Same as item 3 gate No. 1.

5. Same as item 4 gate No. 1.

Gate No. 3 (west gate)

1. The radial bracing at each end was welded to the plates in lieu of riveting due to the trouble encountered in riveting gates Nos. 1 and 2.

2. Three shop rivets were cut out on the third bottom skin plate joint from each end of the gate next to the corner where upstream and bottom skin plates join in making the two slip joints for lengthening the gate.

3. Same is item 1 gate No. 2.

4. Same as item 2 gate No. 1.

5. The two slip joints left for the purpose of lengthening the gate were left unriveted until all of the other skin plate joints were caulk welded and then the holes were reamed for ⅞-inch instead of ¾-inch rivets, and riveted.

The caulk welding was subcontracted by Virginia Bridge & Iron Co. to Welding Engineers Inc. Skin plate joints were electric arc welded and the downstream skin plate joint welds were ground flush with the surface of the plates. All welds were carefully peened in order to avoid distortion of plates or loosening of rivets in adjacent areas. A caulk weld was specified at each end of each gate where the ½-inch end plate and the ⅜-inch reinforcing plate on top of the bottom skin plate intersect. It was found impossible to make this weld because of insufficient working room, and two caulk welds were made on the opposite side where the 6-inch ship channel is adjacent to these two plates.

All welding was done in accordance with the recommendation of the American Bureau of Welding.[21] Inspection of the erection of the gates for the Authority was handled by one man who reported to the field engineer.

Gate length change.—A shortening of the gates during assembly was discovered after gate No. 1 (east gate) was completely assembled and was being checked for leaks in the drum. The gate was found to be almost 1 inch shorter than the specified length although at the time when the gate was completely assembled and bolted up prior to riveting, the gate was checked and found to be almost exactly the right length.

When this condition was first observed, gate No. 2 was completely bolted up, and riveting and welding were in progress; however, the gate hinge anchors had not been grouted. It showed a shortening of 7/16 inch at this time. Gate No. 3 was bolted up completely and riveting had just been started. Although the length was approximately correct at this time, it was evident that it would also shorten. No thoroughly acceptable explanation was ever offered as to why the gates shortened, although the welding of the skin plates may have been responsible.

This shortening tended to shift the gate hinges with respect to the hinge anchors and cause binding at one side of each hinge. In gate No. 1 it was impossible to shift the hinge anchors so the contractor agreed to saw the hubs on the anchors enough on the close side to give acceptable clearances. In gate No. 2 the anchors had not been grouted and it was possible to shift some of them, and others were sawed to give proper clearance. Two joints were left unriveted in gate No. 3 until all other joints had been riveted and welded. Then the gate was jacked to proper length and the upstream and bottom skin plates at these two joints were reamed for 7/8-inch rivets. These holes had originally been punched for 3/4-inch rivets. The downstream skin plate was redrilled between the original holes and the original holes welded shut. Both joints were finally riveted and welded, and when completed the clearance between the hinge and hinge anchor jaws was fairly uniform and the 70° F. length was 99 feet 7 1/8 inches, or approximately 1 3/8 inches short of the designed gate length of 99 feet 8 1/2 inches (clearance between piers 100 feet 0 inch, ±1/8 inch).

Inside surfaces of the drum gates were painted with one coat of bitumastic priming solution followed by one coat of regular bitumastic enamel. Both the bottom and upstream skin plates on the outside of the gates were painted with one coat of outdoor bitumastic priming solution and one coat of outdoor bitumastic enamel. Downstream skin plates were painted with one coat of regular priming solution only, but action of the gate seat seal scraping against the plate has removed most of this and it will be necessary to repaint the gates at regular intervals. The painting was done on the job by TVA forces.

Progress.—Gate erection was started on December 16, 1935, with the arrival of equipment and supplies. The first gate seat casting was placed on December 17 for the east gate and erection of that gate completed on February 26, 1936. The entire job was finished on April 16, 1936. The installed weight of the three gates was 1,279,412 pounds, including hinge and seat anchors and anchor bolts.

[21] 1931 Report of the American Bureau of Welding, Structural Steel Welding Committee.

Outlet works.

Trashracks.—Forms for the trashrack structures were built in place, largely from material that had been cut to size in the carpenter shop. Each lift was 12 feet 6 inches high and included one set of columns and the beams at the top of these columns. Normal procedure was to place column reinforcing prior to erection of the four panel forms making up each column. This step was followed by erection of the beam forms. Scaffolds with landings at the bottom of each beam were carried up inside the structure. These supported the bottom beam forms and served as working levels from which forms were erected and concrete poured.

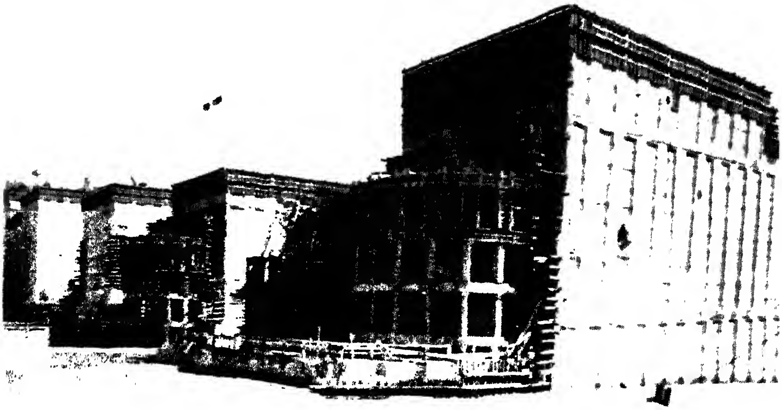

FIGURE 217.—*Trashrack construction.*

Concrete was poured in two lifts. First, the columns were poured, and after a period of several days the beams were poured. Reinforcing steel for the beams and anchor bolts for the trashrack grilles were placed prior to pouring beam concrete. The majority of the concrete was placed using the round controllable bottom dump bucket. Individual chutes were built to direct the concrete into the column forms. A dumping platform was provided at the head of the chutes and the concrete was delivered to the forms in 3-cubic-yard batches. A part of each bucket was divided between several columns. In block 39 a hopper and concrete buggies were used on several pours before the round controllable dump bucket was purchased. The base mat under each trashrack was poured of face mixed concrete with 6-inch maximum size aggregate, 1.10 barrels of cement per cubic yard and a water-cement ratio of 0.58 by weight. A mix containing 1.50 barrels of cement per cubic yard, 1½-inch maximum size aggregate, and a water-cement ratio of 0.55 by weight was used for the rest of the structure. As much as 200 pounds of cement per 3-cubic-yard batch was added at times to improve workability.

Bellmouth sections.—The original design for the inlet end of the discharge tubes utilized a rectangular opening in a vertical plane at the face of the dam. This opening was changed to one utilizing a bellmouth-shaped entrance. This change was made necessary since the entrances at Madden Dam of identical design had eroded. At the time when the revised entrance was agreed upon by the Authority, three of the blocks containing outlet conduits had been poured to levels considerably above the tubes and the entrances had been completed according to the first design. It was therefore necessary to chip away some of the concrete already in place and grout in dowels to anchor the bellmouth concrete to the structure. The fourth block was poured according to the revised plan.

Ribs for the bellmouth forms were fabricated in the carpenter shop of two 2- by 10-inch timbers lagged together and match marked for ready assembly in the field. The lagging which was 1- by 4-inch tongue-and-groove flooring was placed in the field after the ribs had been assembled and matched. In the three blocks which had been poured before it was decided to add the bellmouths, the concrete was poured in three lifts. The first lift included the bottom lip of the bellmouth, the center pier, and the two outside lips for approximately 2 feet above the floor of the tubes. The bottom lip for this pour was completely formed except for a space about 4 feet wide at the top, where lagging was omitted to permit access into the form for vibrating. This surface was finished by screeding. Both outside lips and the center pier up to the bottom of the top lip were included in the second lift. The remainder of the two outside lips, center pier, and the top lip made up the last lift. A space about 30 inches wide at the top of the top lip was left unlagged to allow access for vibrating the concrete. In block 38 the bellmouth was poured monolithic with the dam.

Outlet conduits.—Forms for the concrete section of the discharge tubes were built in place as concreting in the blocks containing the tubes progressed. Ribs were built in place of 2- by 10-inch lumber with the vertical members composed of two boards lagged together, and the top and bottom horizontal members composed of single pieces. These ribs were spaced on about 2-foot centers, and additional rigidity for the vertical members was obtained by using 6- by 8-inch wales. The lagging was composed of 2- by 8-inch tongue-and-groove lumber on all interior faces, except for about 35 feet downstream from the end of the liners, where the floor was not formed. Reinforcing steel was composed largely of straight bars except that at the upstream end around the lined portions of the tubes matched braced bars in two sections completely encircled the tubes. A part of the bottom set of these bars was placed in the piers poured to support the lining casting during assembly and lining up. The straight steel was placed in mats as the forms progressed.

Immediately adjacent to the reinforcing steel around the conduits, concrete containing 1.70 barrels of cement per cubic yard and 1½-inch maximum size aggregate with a water-cement ratio of 0.50 by weight was used. The same mix was used in the concrete for the bellmouths which were added in blocks 39, 41, and 42, and also in block 38 where the bellmouth was poured monolithic with the dam. In the bellmouths for the conduits in blocks 39, 41, and 42, as much

as 200 pounds of extra cement per 3-cubic-yard batch was added to increase workability. Special care was exercised to prevent feather-edged joints where tops of 5-foot lifts intersected the top and bottom of the outlet conduits. Bulkheads 10 inches high placed normal to the conduit form at these points served to produce a joint perpendicular to the neat line of the conduit concrete.

Interior surfaces of the conduits were not finished according to any set plan, except for that portion in each conduit upstream from the liners. In general, the small irregularities were filled with Ironite [22] cement mortar, and after it had hardened all surfaces were given two coats of thinned water-gas tar and one coat of coal tar applied hot. In the tubes of block 38, the first Ironite patches were apparently painted over before the paint hardened, and these patches had to be chipped out and replaced. Surfaces downstream from the lined portion were not treated according to any definite plan, and new ideas were tried out as the work progressed. In most tubes the surfaces were smoothed by grinding high spots. Where honeycomb occurred, the area was chipped out and patched.

In the west tube of block 41 a space 9 feet long and 5 feet high on the east wall was treated with Gar Kem.[23] This area, which had previously been gunited and ground smooth, was wind brushed on the lower half and the upper half lightly rubbed with a carborundum brick. Two coats of Gar Kem were applied, the second about 1½ hours after the first.

Conduit linings.—The linings for the outlet conduits were furnished by the Hardie-Tynes Manufacturing Co. of Birmingham, Ala., on the same contract with the slide gates. Each tube lining was made up of 24 separate castings which matched to form 12 sections of lining. Each was 4 feet long, making a total of 48 feet of lining. Four of the 12 sections form the upstream and downstream frames respectively for the 2 gates.

All parts were shipped to the Coal Creek unloading yard by rail in completely assembled sections of lining 8 feet long. Pieces were unloaded onto heavy-duty trailers for transporting to the job. A single cableway handled the sections to approximate final position in the dam. Concrete piers 6 feet long by 15 to 18 inches wide were poured to support the lining in final position and during concreting. Twelve such piers were poured for each conduit lining, thus affording two piers for each 8-foot section of lining. The entire lining for each tube was assembled, bolted, and shimmed to final line and grade before concreting started. Completed linings were held in place during concreting by turnbuckles anchored in the concrete. Erection was handled by the regular job rigger forces of the Authority.

Slide gates.[24]—Embedded parts of the gates which included, besides the linings, the bonnet, bypass piping, and air inlet piping were installed as concreting progressed. Each gate bonnet, cast in four pieces, was shipped to the job in two sections. These were assembled in the field to make the completed bonnet. The remainder of the assembly which included the gate leaf; the hoist assembly composed

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[22] Non-shrinking cement composed of portland cement and iron oxide.
[23] Protective iron oxide paint.
[24] Specifications for the Norris slide gates are included in appendix G.

of the cylinder, piston, cylinder head, gate stem and bonnet cover; the stem extension; and the gate sills were shipped to the job unassembled. Leaf seats, frame seats, and bonnet seats were assembled to their respective castings in the manufacturer's plant. The procedure followed in assembling the moving and nonembedded parts of a gate were as follows:

1. Gate sill installed.
2. Gate leaf lowered into place.
3. Gate hoist assembly placed and assembled to bonnet. This assembly was received on the job in the assembled condition. The lower leaf nut was attached to the gate stem when received and was removed before the assembly was set to allow the gate stem to be

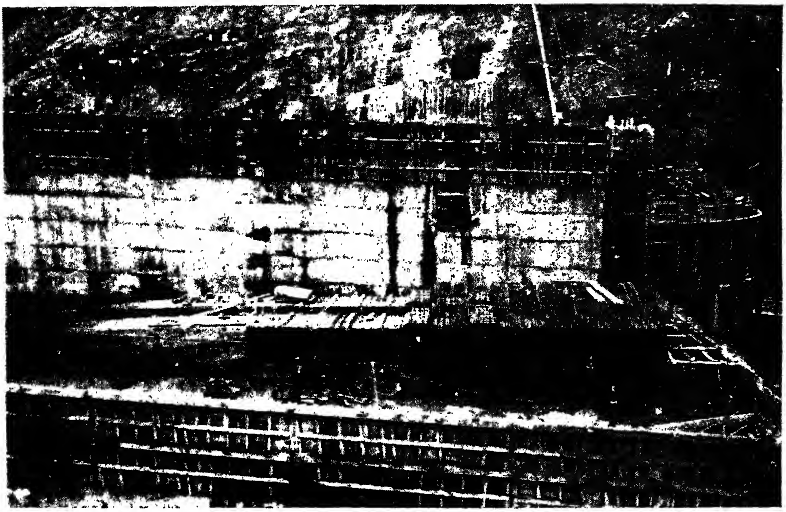

FIGURE 218.—*Linings in place with formwork for downstream concrete.*

lowered into a connecting position with the leaf. The bonnet cover was completely equipped with packing glands, packing gland seats, oil packing, and water packing.

4. Cylinder head removed, valve stem extension placed, and cylinder head replaced.
5. Piston lowered to allow connection of gate stem to leaf. With gate stem in position, the lower leaf nut was placed, connecting the gate stem to leaf.
6. Gate seat babbitt fitted to bottom of leaf by scraping babbitt for full surface contact between bottom of leaf and sill babbitt.

The remainder of the assembly consisted of installing the oil piping and controls, oil pump, oil reservoir, and air vent piping. At the time when the gate hoist assemblies were brought into the operating chambers, the concrete in blocks 40 and 43 was below the elevation of the 880 gallery. The hoists were handled by cableway through

the open end of the 880 gallery in either block 40 or 43 depending on which was nearer to the chamber in which the equipment was to be installed.

A drum hoist located in the gallery was then used to pull the assemblies into the gallery as the cableway lowered away. This hoist was also utilized in setting the assemblies in final position. A temporary operating unit which included an oil pump on skids and temporary piping to the hoist was used for operation of the gates before the permanent oil piping and oil pump were installed. The gates were raised and lowered during erection by admitting air into the hoist cylinders through temporary pipes. Installation was handled by the regular job mechanical forces.

Progress.—Assembly of the liners for the outlets in blocks 39, 41, and 42 was done between October 22, 1934, and February 4, 1935. Since block 38 served as a river diversion opening, erection of the liners for the four pairs of outlet conduits was delayed about 4 months. These were assembled and concreting started on June 21, 1935. Gate erection began in November 1934 and was completed before March 4, 1936, the date on which the gates were closed and the impounding of water started. Liners, gates, and bonnets for the eight outlets weighed approximately 2,030,490 pounds; and the operating mechanism, including the gate stem, bonnet cover, and hoist weighed approximately 533,000 pounds, including 35,000 pounds of oil for operation. Erection of the liners and the installation of the mechanical equipment for the gates were handled respectively by the job riggers and job mechanical forces.

SPILLWAY BRIDGE

Steel for the spillway bridge was fabricated at the plant of the McClintic-Marshall Corporation in Chicago, Ill., and shipped by rail to Coal Creek, Tenn. Girders were fabricated complete with gusset plates for lateral bracing connections and floor beam connecting angles attached but not completely riveted. The floor beams, stringers, diagonal bracing, expension joints, and other parts were fabricated separately.

The size of the girders, particularly with regard to length, presented a difficult handling problem. Each girder was 107 feet long, 7 feet 1 inch deep, and weighed 40 tons. They were shipped in pairs, each pair being loaded on three railroad flat cars, and were transferred to the heavy-duty float trailer by means of the special double A-frame tower built to span the railroad siding at Coal Creek. Girders were moved to the west end of the dam over the freeway and down the west side road past the mixing plant and out onto the concrete transfer trestle. From this position both cableways, working in tandem, picked up a single girder and moved it to its position on the spillway bridge. For this work, a lifting beam weighing 6,800 pounds was used. It consisted of a 36-inch, 175-pound section I-beam, 29 feet 6½ inches long. At either end of the lifting beam, 6- by 1-inch bars were set to catch the shackles from the cableway fall block. At the beam center was hung a 4-inch eyebolt with a nut swiveling on a roller bearing made of ⅞-inch hard steel balls. This swivel turned in a bolster which was clamped to the beam with

four 2-inch rods. In order to strengthen the beam against lateral deflection, four 1-inch rods formed a truss at the top and bottom flanges.

As an aid to the signal man in keeping the beam level, a 30-inch pendulum and a target were attached near the center of the beam. One signal man controlled both cableways. It was necessary to plug in his telephone at three different points during the travel of the load. About 4 hours were required to transfer the first girder, since

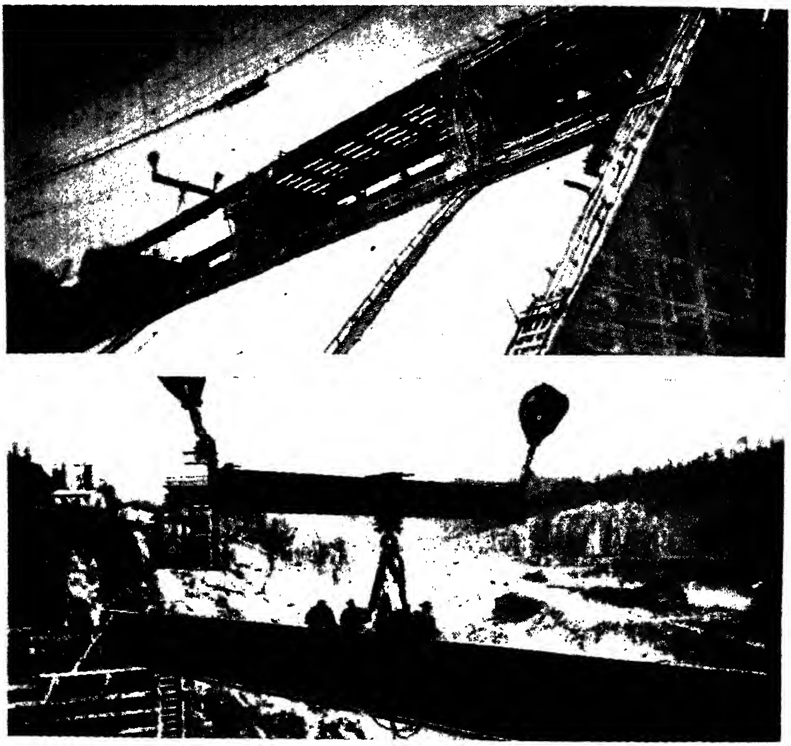

FIGURE 219.—*Placing girders for spillway bridge.*

some time was lost in checking the deflection of the main cables of the cableways and maneuvering the trailer on the concrete delivery trestle. The remaining girders required approximately 2 hours each to place.

Accepted practice and the specifications were followed in placing and riveting the steel. Rivets were inspected by a member of the field engineer's organization. A total of 4,806 steel rivets was driven, of which about 4 percent were rejected, cut out, and redriven. The steel was given one shop coat of red lead except on machine-finish surfaces and those which would be in contact with concrete. After

the steel was erected and the concrete placed, all exposed steel was given one coat of aluminum paint after all field rivets and damaged surfaces had been given a patch coat of red lead.

Concrete.

Concrete for each bridge unit was poured in four lifts. First the roadway slab, second the two sidewalks, and third and fourth the girder encasement which formed the parapet walls. Parapet walls at the piers and abutments were poured independently of the bridge concrete.

Bottom forms for the roadway slab were supported by a series of horizontal ribs, spaced on about 15-inch centers, built up of 2- by 8-inch lumber placed between the webs of adjacent stringers. Each end of the ribs was supported on the lower flange of the stringer beam by two vertical members, under which wedges were placed to secure the ribs in place to proper grade. Lagging was 7/8- by 8-inch matched dressed lumber. Structural steel "end dams" served as vertical bulkheads at the ends of each bridge section; and two other bulkheads, one at each side of the roadway slab, completed the forms for the bridge deck concrete. Bridge deck concrete was composed of 1½-inch maximum size aggregate, with 1.50 barrels of cement per cubic yard and a water-cement ratio of 0.55 by weight. The concrete for the east and center spans contained 200 pounds of extra cement per 3-cubic-yard batch while 400 pounds of extra cement per batch was added for west span concrete. The bridge deck was left about 2 inches low and the surface rough screeded since it was to be surfaced later with natural rock asphalt.

After the bridge deck was completed, the sidewalks were poured with bottom forms similar to those for the bridge deck. A girder formed the side bulkhead on one side of each sidewalk, and a wooden bulkhead which rested on the bridge deck concrete confined the concrete and formed the curb at the edge of the roadway. The structural steel "end dams" formed the bulkheads at the ends of each bridge span. Concrete for the sidewalks was of the same mix as used for the roadway except that 100 pounds of cement per 3-cubic-yard batch was added to this mix. Sidewalk surfaces were given a wooden float "pulled" or "gander" finish.

Parapet walls and girder encasements were poured in two lifts—the first including the bottom 5 feet 3½ inches of the outside girder encasement. The second lift included the remainder of the outside and inside girder encasement and formed the parapet wall.

Forms for the first lift were constructed from a scaffold suspended from the girders. This scaffold consisted of a walkway and railing supported on 6- by 6-inch oak stringers which were hung from 6- by 8-inch oak outriggers resting on top of the girders. Stringers were suspended outside the girder by ¾-inch steel rods and inside of the girder by heavy twisted form wire. Outriggers were spaced about 8 feet on centers and were braced on the inside of the girders to add rigidity to the set-up.

The outside forms proper for the girder encasement were built up of 6- by 8-inch wales on the 3-foot 6-inch centers, 2- by 8-inch studs on 15-inch centers and 7/8-inch unmatched dressed lagging. Panels of studs and lagging were made up on the bridge deck in 8-foot lengths

and placed by hand. Wales and anchor bolts were placed after all panels in one section were in position. Anchor bolts for top and bottom wales were removable hook clamps secured to the top and bottom girder cover plates. For the center wale, anchor bolts were spot welded to the girder web stiffeners.

Inside parapet forms were made up of single panels built on the bridge deck and moved into place. These panels were made of 5/8-inch-thick matched tongue-and-groove flooring, placed on a layer of 2-inch dressed lumber. The lagging was placed horizontally on vertical studs, and the whole panel was constructed to conform to the camber of the bridge girders. Bracing was obtained by means of 6- by 8-inch struts between the top and bottom of panels and a sill on the roadway surface held in place by dowels in the roadway concrete. Alternate struts were placed at the bottom and top of the panel. Wooden spacer struts on the inside of the panel between the form and bridge girder were removed as concrete was placed. Forms were carefully scraped and sawed to remove all irregularities so that no perceptible form marks would appear on the finished concrete.

Concrete was delivered to the forms in the square controllable dump bucket in 2½- and 1¼-cubic-yard batches and consisted of ¾-inch maximum size aggregate, 1.70 barrels of cement per cubic yard, and a water-cement ratio of 0.55 by weight with 100 pounds of cement per cubic yard added at times to improve the workability of the mix. Because of the small clearances between the forms and the structural steel and the limited space through which to place the concrete in the forms, particularly on the outside of the girder, considerable grout was also used. It was composed of coarse sand as the maximum size aggregate, and contained approximately 2.80 barrels of cement per cubic yard with a water-cement ratio of 0.53 by weight. The top of the parapet was given a wooden trowel finish after screeding to proper shape.

The parapets at the abutments and two spillway piers were poured separately from the bridge parapets. These forms were built in place using the same type of material as for the other parapet walls. Circular ribs for the upstream pier parapet walls were fabricated in the carpenter shop and the ribs were used horizontally with vertical 2-inch dressed lagging and 6- by 8-inch wales. Downstream pier parapet forms were straight and were built in place. Concrete for the pier parapets was of the same mix as used for the bridge parapet walls.

ROADWAYS

In the nonoverflow sections, the upstream face of the dam was formed with the regular upstream panel forms to elevation 1,055. Above this point the parapet walls were formed with vertical built-in-place forms offset 7 inches from the upstream face. For the downstream face the regular sloping panel forms were used to elevation 1,034.42. From this point to elevation 1,053, vertical forms were built in place of 2- by 8-inch studs on 15-inch centers, 6- by 8-inch wales on 2-foot 6-inch centers and 2-inch dressed lagging. A special cantilever form was constructed for the concrete above elevation 1,053. An 8-inch step had previously been formed at elevation 1,033.5 for the entire length of both abutment sections to provide a supporting seat for these cantilever forms.

Forming of the cantilever section above elevation 1,053 presented a difficult problem, and after considerable study the special cantilever form was designed. Four 30-foot form sections, or two sets, were built. Each form weighed 15 tons and was similar except that the two in each set were so constructed that they mated at the center line of a block and permitted forming even the longest blocks by two sections. A single cableway was used for each section during the setting and moving from one point of use to another. A four-way sling was devised to facilitate handling. The following steps were required to remove and reset each section of form:

1. All sleeve bolts were removed except a few to hold the form in place and two lines of the cableway sling were hooked on.

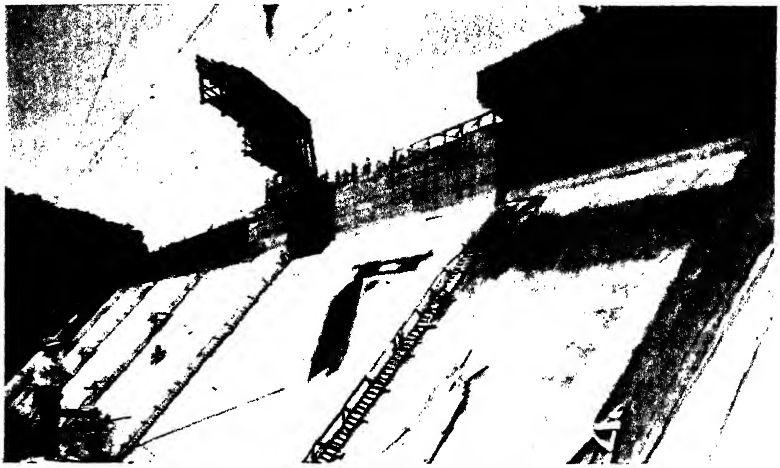

FIGURE 220.—*Cantilever forms were handled by cableway.*

2. The remaining sleeve bolts were removed and the cableway moved downstream enough to loosen the form and swing it free.
3. After lowering to the ground the remaining lines of the sling were hooked to the form which was then picked up by the cableway and assumed an upright position.
4. The form was moved to the new location and placed in an approximate position on the supporting shelf. It was then moved into exact position by hand and enough sleeve bolts connected to hold the form safely in place.
5. The cableway was released and the remainder of the form erection completed.

Outside parapet forms were built in place of 2-inch dressed lagging, 2- by 6-inch studs on 15-inch centers and one 6- by 8-inch wale at the top and bottom of each form. Inside parapet forms were built in panels 28 feet long and each panel was reused about six times. These forms were somewhat similar to those used on the spillway bridge parapet walls that have been described in the previous section.

Surfacing and drainage.

The roadway across the abutment sections of the dam and the spillway bridge is surfaced with Kentucky rock asphalt averaging about 3 inches in thickness at the crown of the road and about 2 inches at the curbs. This work was done in accordance with the Tennessee State Highway Department specifications for asphalt paving. A natural sandstone rock asphalt was used and 449.2 tons were required to complete the surfacing. Before the asphalt was placed, the low places in the concrete surface were filled with bituminous concrete to smooth the subgrade and insure a uniform thickness, and the entire roadway surface was primed with an emulsified primer. The asphalt was placed in two layers using a blacktop paver for distributing and screeding.

SWITCHYARD

The necessary earth excavation was handled largely by a 1¼-cubic-yard shovel and a fleet of rented trucks. Most of the rock excavation was handled by the cableways in skip pans which were loaded by hand and the excavated material deposited behind the gravity spillway training wall. Earth excavation was deposited in the fill for the parking area downstream from the powerhouse on the east bank of the river. Quantities involved were 11,376 cubic yards of earth and 6,034 cubic yards of rock.

Concrete.

All equipment and structural steel for the switchyard rests on reinforced concrete footings anchored to rock by means of reinforcing steel bars grouted in wagon drill holes. The cable tunnel, retaining walls, fence supports, and walkways are also of reinforced concrete construction. Forms for these structures were built in place, following the usual job practice in construction of forms for small concrete structures. The cable tunnel furnished the largest single item of concrete work in the yard. Forms for this structure were built in two lifts while concrete was poured in six lifts. Concrete for the base slab and floor of the tunnel made up the first concrete lift. The first lift of forms was used for the second and third lifts of concrete and included the side walls and intermediate slab, respectively. The second lift of forms confined the upper walls, top slab, and curb. Premolded asphalt contraction joint fillers were placed in all contraction joints.

The majority of the concrete for the foundations and footings and all of the cable tunnel and pull vault concrete was mixed, transported, and placed by the regular facilities, using the square controllable dump bucket for placing. Concrete for the transformer transfer tracks and structural steel footings on the low level of the switchyard was mixed in a small portable mixer which was put into service after the central mixing plant was dismantled. This mixer was placed immediately downstream from the powerhouse on the same level with the low level of the switchyard, and concrete buggies were used to transport mixed concrete to the forms.

Three classes of concrete were used in the foundation and footings which were completed before the central mixing plant was dismantled. In the large forms, where the reinforcing steel was not

too thick, concrete was composed of 3-inch maximum size aggregate with 1.33 barrels of cement per cubic yard and a water-cement ratio of 0.55 by weight. In the heavily reinforced sections, a mix was used of 1½-inch maximum size aggregate, 1.50 barrels of cement per cubic yard and a water-cement ratio of 0.55 by weight. Extra cement up to 200 pounds per 3-cubic-yard batch was necessary at times to improve workability because of the prevalence of dirty aggregates near the end of the job. The third class, used in the cable tunnel walls, was composed of ¾-inch maximum size aggregate with a water-cement ratio of 0.55 by weight and cement contents varying from 1.90 barrels to 2.25 barrels per cubic yard.

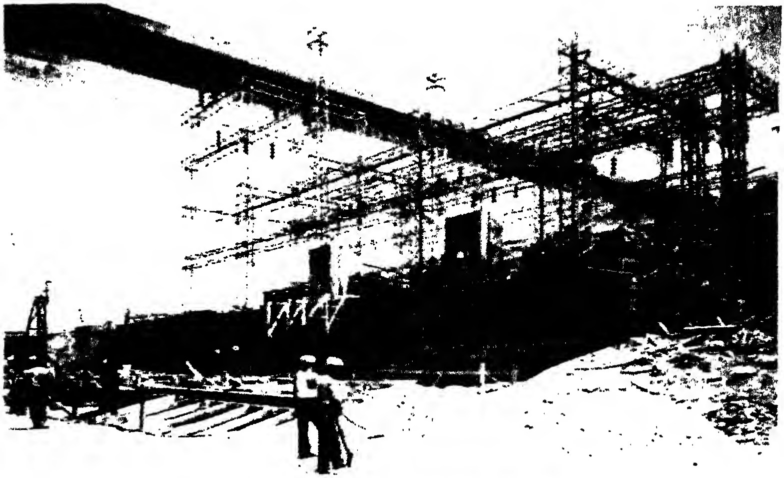

FIGURE 221.—*Switchyard during construction.*

Steel.

Switchyard structural steel members are galvanized and bolted together. They were shipped to the job unassembled, with all members punched and ready for assembly. The laced columns and beams were assembled into convenient units for moving into final position and handled into place by cableway. Erection of the structural steel was done by the regular job rigger forces. Approximately 131 tons of steel made up the initial switchyard installation which was completed between June 6 and 30, 1936. Switchyard extensions made after this installation are not covered in this report.

Equipment.

The transformers were shipped by railway freight from the manufacturer's plant at Pittsfield, Mass., to Coal Creek where unloading and transfer to the powerhouse were handled by the rigger forces of the Authority. The item presenting the principal problem of unloading and transfer was the transformer case containing the core and coils. This case, which weighed 81,000 pounds, was picked

up from the car and placed on the 40-ton float trailer by the double A-frame unloading rig and securely fastened to the trailer by blocking and guys. Two 8-ton trucks heavily loaded, one pulling in front of the trailer and the other braking from behind, provided the motive and braking power for the trailer in transit from Coal Creek to the powerhouse. The transformers were carried down the winding west bank construction road, which had three hairpin curves, one of which had a radius of less than 50 feet and all of which were

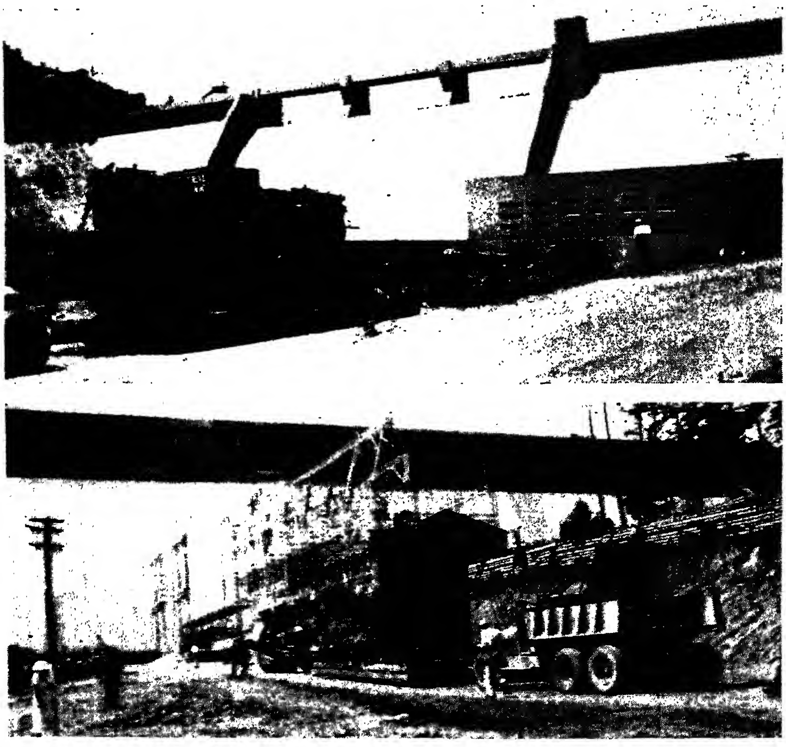

FIGURE 222.—*Moving transformers from Coal Creek to the switchyard.*

steeply superelevated. Special care was necessary to prevent overturning of the trailer and transformer at these points. To accommodate this heavy load, it was necessary to reinforce the construction bridge. Six transformer cases were unloaded and transferred to the powerhouse without mishap or damage although because of their height and weight they presented the most hazardous handling problem attempted during the job. At the powerhouse the trailer was backed through the equipment entrance onto the unloading bay, where the transformers were picked up by the powerhouse crane and transported to the erection bay at the west end of the powerhouse.

Transformers were shipped to the job completely closed and filled with nitrogen gas to protect the coils and core from atmospheric effects. The tank of each transformer was flanged 4 feet below the top and the top 4 feet removed to reduce the height for railroad clearance. General Electric Co. personnel handled the entire field assembly, which included placing on the trucks, adding the 4-foot top section, assembling the bushings and radiators, filling with oil and nitrogen, and assembling all accessory equipment. After assembly and drying out, the transformers were placed by means of the powerhouse crane and the transformer transfer equipment. A small amount of oil was placed in each transformer before it was moved into final position in the switchyard, and filling with oil and nitrogen gas was completed after the units were placed in their final positions. Oil for the transformers was also furnished by the General Electric Co. and shipped to the Authority in tank cars and drums.

The first transformer was received on the job in June 1936, and the erector crew arrived June 22, 1936. The first bank was completed and placed into service on July 28, 1936, and the second bank on September 30. Additional equipment necessary for completion of the switchyard presented no difficulties in erection or transportation and were installed either by the contractor or by the Authority's erection forces.

LOYSTON DIKE

At a point about 5¼ miles east of Norris Dam, a saddle dam was built to prevent the loss of flood storage water above elevation 1,036 from Norris Reservoir through a low point in the reservoir rim. The location of this dam is shown in figure 225, page 486.

Since the low point of the saddle was at elevation 1,036, water would have escaped down Buffalo Creek into the Clinch River below Norris Dam whenever it became necessary, for flood control purposes, to fill the reservoir above that elevation. The character of the underlying rock was also a matter of concern because of the indication of the presence, in the immediate neighborhood, of extensive underground channels which might allow water to escape through the reservoir rim below elevation 1,036. In order to prevent leakage under the dike, a program of grouting was carried out and an impervious core to connect the fill with impervious material on either side was constructed.

The original plans contemplated a rolled earth embankment with the crest at elevation 1,060. The structure was substantially completed as first contemplated, but a later decision regarding the safe height resulted in increasing it an additional 5 feet, bringing the top to elevation 1,065. As finally completed the structure is 10 feet wide at the top, with an upstream slope of three to one and a downstream slope of two to one. The maximum section is approximately 32 feet high above the original ground and approximately 170 feet wide at the base. Beginning at the west end where it abuts a hill forming the west side of the saddle, the dike extends eastward across the low point in the saddle for approximately 1,280 feet, measured along the crest, and intersects a spur from the hill forming the east side of the saddle. At the intersection with the spur, the axis turns an angle of 37° to the left for a distance of 14 feet, from which point the dike

extends an additional 700 feet in a northeasterly direction along the spur to original ground. A cut-off trench of impervious material placed in the same manner as the rolled fill extends from the original ground at least 5 feet into impervious material or to rock. The trench extends only along the lower part of the saddle, beginning at the west end of the dike and extending to station 17+00.

By the time the decision was reached to increase the height 5 feet, the riprap for the protection of the upstream slope had been completed and the fill necessary for the additional 5 feet of height was placed on the downstream side of the dike. Thus the center line of the 10-foot berm on top was moved 15 feet downstream from the

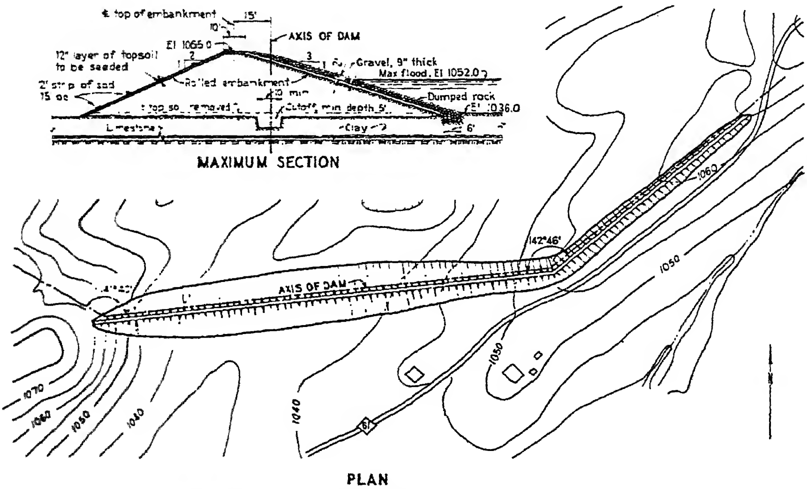

FIGURE 223.—*Loyston dike—Plan and elevation.*

original center line of the dike. The upstream slope is protected from erosion by wave action or surface water run-off by a blanket of riprap approximately 3 feet thick. The top and downstream slope are protected by a growth of small grasses and vines.

Construction organization.

Construction of Loyston Dike presented no unusual problem. The scheme of operation contemplated the use of personnel and equipment from Norris Dam in such quantities and at such times as the main construction program would permit; but, in general, once work at the dike had been started it was carried to completion in an orderly and efficient manner.

With the exception of the grouting program, which was under the supervision of the regular foundation treatment organization, the construction organization was under the supervision of one general foreman who had complete charge of all construction activities. One man, who served as resident engineer, handled all engineering and inspection. He was assisted in lay-out work by a field party,

from the main field engineering organization and in making soil tests by members of the concrete inspection organization. The construction organization included, besides the general foreman, three labor foremen, four shovel operators, eight tractor operators, four shovel oilers, four compressor tenders, four core drill operators, four core drill helpers, four wagon drill operators, one pump operator, and one timekeeper clerk. The remainder of the organization was composed of unskilled labor used for clearing and grubbing and for placing riprap and sod. All hauling was by contract, and the contractor furnished the labor for truck operation and maintenance.

Work was carried on during four 6-hour shifts, except for about 2 weeks at the beginning of operations during which time only two shifts were employed. A two-shift basis was resumed after the fill for the dike, as originally designed, was completed. During the period when the dike was being raised 5 feet, work was carried on during two 6-hour shifts.

Clearing and stripping.

Clearing and grubbing were started in April 1935. The site was stripped of top soil and sod for a depth of about 1 foot. This material was stock piled for later use in sodding the downstream slope and the top of the dike. All clearing and grubbing were done by hand with the aid of tractors for removing large stumps. A tractor, equipped with a bulldozer, loosened and concentrated the stripped material for handling by the 1¼-cubic-yard shovel into rented trucks for transportation to the stock pile area.

Trench excavation.

After completion of the stripping operation, the shovel excavated the trench for the impervious cut-off or core. This trench extended from the west end of the dike for 1,200 feet to station 17+00, was 12 to 15 feet wide at the bottom and from 20 to 25 feet wide at the top and at least 5 feet deep. It was not excavated to rock except where rock was less than 5 feet below the original ground surface. At other points it was extended into the practically impervious clay blanket overlying the divide. Suitable material removed from the trench was placed directly into the fill, and the remainder was wasted on the upstream side of the dike beyond the boundary of the structure. The original plans called for the center line of the trench to coincide with the axis of the original dike, and the trench was completed in this location. In the completed structure the center line of the trench is 15 feet upstream from the center line of the 10-foot berm on top of the dike. All excavation was done by means of the shovel loading into rented trucks for transporting to either the fill or spoil banks. Excavation for the core trench amounted to 4,142 cubic yards of earth and was completed by mid-April.

Grouting.

The original grouting plan contemplated drilling all grout holes on the center line of the cut-off trench; but field conditions dictated several changes. At the beginning of operations, wagon drill holes on 20-foot centers were drilled between stations 13+40 and 16+80

on the axis and at the bottom of the cut-off trench. Intermediate wagon drill holes halfway between the first holes were later drilled in this area. This section was then grouted through these holes without any attempt at washing seams or establishing interconnection between holes. From station 17+40 to station 23+80, 3-inch core drill holes spaced 20 feet apart were drilled on the axis and grouted before the fill was made. It became apparent as grouting progressed that if the section between the west end of the dike and station 13+40 was to be grouted from holes on the axis drilled from the bottom of the cut-off trench, this work would interfere with the construction of the impervious core and consequently hold up work on the rolled fill. In view of this fact, a decision was reached to grout this section of the divide from holes drilled at the toe of the upstream slope. Complying with this decision, 3-inch core holes on 20-foot centers were drilled to ground-water level and were subsequently grouted. Three 3-inch core holes were drilled and grouted to connect the line of wagon drill holes on the axis with the line of core holes on the upstream toe and thus insure a continuous line of holes for the length of the dike. These core holes were washed just enough to establish interconnection between them.

Grouting pressure in general was limited to 15 pounds per square inch, but in some cases final grouting pressure reached 30 pounds per square inch. All holes were water tested for leakage at pressures up to 20 pounds per square inch before being grouted. Grout consisted of one part of portland cement and one part of water by volume, except in two or three cases where holes were started using 1.2 parts of cement to one part of water. The procedure followed in grouting was the same as that for the shallow grouting of the dam foundation.

All drilling and grouting were done between mid-April and early August 1935. The work done in this operation is summarized below:

	Quantity	Unit	Unit cost
Wagon drill holes	1, 541	Linear feet	$0. 853
Core drill holes	2, 443	do	1. 690
Grouting	5, 398	Cubic feet	1 849

Rolled fill.

Except for a small amount of material from the core trench, material for the rolled fill and cut-off core wall was obtained from two borrow pits. The first pit was located about 300 feet north of the dike on the property owned by the Authority, the average haul being about 500 feet. However, since this pit proved inadequate to supply the total amount of material needed for the fill, a second borrow pit known as the York pit was opened and supplied the major portion of the material used for the dike. This pit was located approximately 1,700 feet from the west end of the dike with the average haul being about 2,600 feet.

Borrow pits were stripped of all top soil which contained roots, sod, and perishable matter of any kind; and this material was

stock piled together with the top soil stripped from the site of the dike. All stripping and excavation for borrow was done by the 1¼-cubic-yard shovel and the excavated material was handled to the stock pile or fill by a fleet of from 10 to 15 rented trucks. Material for the fill was dumped by the trucks in approximate position and spread into layers, which when compacted did not exceed 8 inches in thickness. Tractors equipped with bulldozers did the spreading while compaction was done by sheepsfoot rollers and a 10-ton road roller.

Each layer of material was placed to extend outside the line of the finished structure, and the slopes were trimmed to finished line by a tractor-drawn grader. The excess material from the trimming of the slopes was in turn redeposited in the fill. In order to allow for settlement, the depth of fill at all points was made approximately 5 percent greater than was necessary to bring the structure to the designed neat lines. The area of contact between the original ground and the fill was scored from 6 to 8 inches deep by plowing on 3-foot centers before the placing and compacting of material was started. In places where the road roller was used, the surface of the compacted layer was scored lightly by a scarifier before additional fill was placed, and spikes were used on the road roller to minimize the marked laminations. All of the material placed in the addition to the original fill was completed by sheepsfoot rollers.

Work on the original fill was started on April 4, 1935, and was completed on July 25, 1935. The additional work necessary to raise the top of the dike 5 feet in elevation was started on September 17, 1935, and was completed December 20, 1935. All material was handled and placed in the same manner as has been previously described in placing the main fill. A total of 80,458 cubic yards of material makes up the completed fill. Of this amount, 22,500 cubic yards were required for the additional 5 feet of elevation. The total yardage was placed at a cost of $0.642 per cubic yard.

Compaction test.

Proctor's method[25] for controlling the compaction of rolled earth-filled dams was used to check the moisture content and degree of compaction obtained in the fill. A very close control over the degree of compaction was not attempted, and therefore, the testing was not carried out either on a regular schedule or in a very extensive manner. Tests included determination of moisture content in the borrow pit and in the fill, degree of compaction in the fill, plasticity in the fill and laboratory-compacted materials, and laboratory compaction to determine the optimum moisture content and the degree of compaction obtainable. Samples taken from the fill and both borrow pits indicated an average moisture content in place of about 28 percent. Additional samples indicated the average optimum moisture content to be approximately 21 percent and laboratory maximum compaction to give an average dry weight of 104 pounds per cubic foot. Under best laboratory compaction at existing or field moisture in the fill, an average dry weight of 96 pounds per cubic foot was obtained. Samples from the fill showed

[25] Proctor, R. R., Rolled Earth Dams, Engineering News-Record, vol. 111, pp. 245–248, 286–289, 348–351, 372–376.

an average dry weight in place of 89 pounds per cubic foot. Although the tests were not extensive, they indicated that the moisture content of material placed in the fill averaged 7 percent too high for maximum compaction and field compaction averaged about 7 pounds per cubic foot less than would have been obtained at the existing moisture content with best compaction.

Erosion protection.

Protection of the upstream slope against erosion from surface water run-off and wave action was by a 3-foot layer of quarry rock riprap laid directly on the slope. The original plans called for a gravel blanket 8 inches thick, but the high added cost necessary for buying and transporting this material from the aggregate supply at the dam did not seem to warrant this part of the riprap. Rock for

FIGURE 224.—*Upstream slope of Loyston dike.*

the riprap was taken from a quarry near the borrow pit fom which the earth fill was obtained. Quarry-run material was dumped on the slope in approximate position and placed by hand. During the hand-placing process, as much of the small material as possible was worked to the bottom of the riprap partly to fulfill the requirement of a gravel blanket. A trench 6 feet wide and about 3 feet deep was provided at the intersection of the earth-fill slope and natural ground surface to form a footing for a riprap.

The top of the dike and the downstream slope are protected from erosion by a growth of small grasses comprised of lespedeza, orchard grass, rye grass, clover, and red top. In addition to the grasses, a growth of Kudzu vine aids in preventing erosion. Preparation of the surface of the dike for these growths was secured by spreading the topsoil over the surface to an approximate depth of 12 inches. A mixture of rock flour, reclaimed from the concrete aggregate manufacturing plant at the dam, and TVA triple phosphate in the proportion of 5 to 1, was applied to the topsoil surface at the rate of

about 1 ton of the mixture per acre. After this application, grass seed was sown at the rate of 20 pounds per acre and the Kudzu roots placed on about 2-foot centers. The entire surface was then covered with a mulch of straw. The downstream slope and top are surrounded by a 5-foot wire fence to keep animals from trespassing.

CONTRACTION JOINT GROUTING

Provision is made for the future grouting of the dam contraction joints. The grouting program will be done at a time when the internal concrete temperature will be within 4° F. of the final estimated temperature. Present indications are that this program can be carried out any time after the spring of 1940. Beyond this date the rate of heat transfer will be very slow.

The contraction joints are approximately 56 feet apart. Each vertical joint is broken up into 50-foot lifts by means of horizontal grout stops. Each lift contains one 1½-inch grout supply header at the bottom and one 1½-inch return header at the top. The two headers are interconnected by ½-inch vertical riser pipes spaced at 5 feet on centers. Grout cells are placed in the vertical risers at 10 feet on centers and so arranged that they are staggered 5 feet on centers with adjacent risers. A typical arrangement of the grout cells is shown in figure 30.

In each 50-foot vertical section of contraction joint the piping arrangement is approximately identical, variations from the set plan being due to some shape or other characteristic of the structure not common to all sections. In general two 1½-inch headers, placed on a 5 percent upward slope from upstream to downstream face, are provided in each 50-foot section. The headers and the grout stops were placed on a 5 percent slope to conform to the slope on which the horizontal construction joints or "day's work" planes were made. The majority of the pipe was bought cut to length and threaded for installation in vertical lifts of 5 feet, although a small amount was bought in random lengths for cutting and fitting in the field to care for special situations.

Grout cells.

Grout cells were made of two covers of 4-inch diameter conduit outlet boxes faced together with the flanges matched. One-half was embedded in the concrete of one block and the other half in the concrete of the adjacent block so that, when the contraction joint opened, a space between the flanges of mating covers was formed through which the grout may flow into the joint.

Grout piping and the conduit outlet covers to which the pipe connections were made were placed first as the high block was carried up. The covers were assembled to the ½-inch nipple and secured to the nipple by means of a lock nut on the outside and a capped conduit bushing on the inside. The capped bushing was used to preclude any possibility of grout getting into the ½-inch pipe during concrete placing. After the forms were removed, the bushing was replaced by another ½-inch lock nut. Several methods of securing the covers to the forms were tried. The method most extensively used on ap-

proximately half of the cells employed a combination of nails and $\frac{3}{16}$- by $\frac{3}{4}$-inch machine bolts with hexagonal nuts. In addition to the fastening for the cover to the forms, the pipe was secured in place and held plumb during concrete placing by means of wooden block spacers fastened to the form to which the pipe was temporarily attached. As concreting progressed the spacer blocks were removed.

As the low blocks were concreted, the matching halves of the cells were fastened and held in place by the nails and machine bolts placed with the previously embedded half of the cell. The edges of the two covers at the point of contact were sealed against leakage of grout into the cell during concreting by means of an application of stiff asphaltic caulking compound. Laboratory experiments at Norris and actual installations at other jobs indicated that this method of treating the cover openings was the most practical.

Grout and water stops.

Nine-inch-wide copper grout stops at the downstream face and 14-inch-wide copper water stops at the upstream face were carried up with each 5-foot concrete lift with enough additional material projecting above the lift to permit connection to the next piece above. Horizontal stops, which were placed at 50-foot intervals, were placed just below the top of a 5-foot lift.

Connection joints between strips were riveted and soldered to form a grout tight connection. The 9-inch stops were bent at a 90° angle along the center line and one 4½-inch leg laid flat against the form of the high block, the other extending into the concrete of the lift. As the low block was built up, the remainder of the stop was bent out so as to be included in the concrete of the lower lift. Similar procedure was followed on the 14-inch water stop except that it contained a 1-inch crimp along the center line of the strip to provide for any movement of adjacent blocks. Horizontal grout stops were handled in a manner similar to the downstream face grout stops.

Provisions were made for grouting at points where the dam abuts against rock in a plane which lies approximately normal to the axis and diverges from the horizontal enough to allow the possibility of the formation of a shrinkage crack at the plane. Water stops at the upstream face and grout stops at the downstream face embedded in rock and conrete across the plane were provided. Grout cells were also installed by placing half of a regulation grout cell—one conduit outlet cover—against the rock and were sealed against the entrance of grout during concrete placing by an application of stiff asphaltic caulking compound. The piping was carried into the nearest convenient gallery.

In the first two or three blocks concreted, a grout stop was provided near the rock foundation, roughly parallel to the rock across the contraction joint. In all subsequent blocks this stop was omitted and the upstream water stop and downstream face grout stop extended into rock, thus providing for the passage of groupt to foundation rock. These stops were anchored in the rock by drilling several jackhammer holes in a line as close together as possible and breaking out the web between holes to make a trench to fit the copper sheet. The sheets were then grouted in the trench which was usually about 12 inches deep.

Contraction joint data.

Before concrete placing was started the chances of obtaining groutable joint openings were considered in view of eliminating the contraction joint grouting system. The limitation of difference in elevation between adjacent blocks as specified in the general specifications was also given consideration. It was decided that on the basis of previous knowledge groutable openings would probably be obtained. Some engineers were of the opinion that if the sides of high blocks with large differences in elevation between high and low blocks were exposed for long periods of time, these joints might not open enough for grouting. Readings of the joint meters installed in the joint between blocks 39 and 40 and all joints west of the 39–40 joint indicate that groutable openings will be obtained.

Contraction joint leakage.

Indications of water having entered the contraction joints from the reservoir and found its way to the downstream face were evident in a number of places. At several joints, entrance of water from the reservoir into the joint has been evidenced by the discharge of water from the contraction joint grout pipe system at the downstream face where pipes were unintentionally left unplugged.

Conclusions.

As a result of the experience gained in providing for grouting of the contraction joints several conclusions are evident:

1. Of the several methods tried for securing grout cells during concreting, the method used most extensively was the most satisfactory. A better method is desirable since none of those tried were entirely satisfactory.

2. Water and grout stops could possibly be installed using keys. There are some disadvantages to this method of installation. However, the necessity of working on each joint twice would be eliminated. Brazing of joints is probably better than soldering, although riveting should also be used in either case.

3. Present indications are that groutable openings will be obtained in practically all joints. It is desirable that the construction program be so planned as to eliminate long exposures of contraction joint faces. It might prove profitable in a dam as large as Norris to install at least three contraction joint measuring devices across each contraction joint at carefully selected positions. Such an installation would provide data on the size of openings and would also prove profitable during grouting in determining the movement of the blocks as the grouting progressed.

4. Leakage into contraction joints from the reservoir is almost inevitable, although it will be accompanied by the depositing of silt and material leached from the concrete which will tend gradually to reduce this leakage. Since leakage is inevitable, provision should be made for intercepting it by means of a vertical drain across the joints a short distance downstream from the water stop.

5. If a contraction joint grouting program is to be attempted, some means, such as the installation of electrical resistance thermometers, should be provided for determining the progress of cooling in

the interior of the mass as a basis for intelligently predicting the proper time for grouting. Recent developments of mathematical formulae have made it possible to predict with a decided degree of accuracy the temperature phenomena of a mass of concrete. However, a carefully planned installation of temperature detecting devices would aid considerably in the use of these formulae.

6. Relief vents should be provided to allow the escape of air and water during grouting.

7. Vertical water stops should be wider than those used and they should be spaced farther from the upstream face. Also the expansion due to opening of contraction joints should be provided for by means other than the V bends used. The stainless steel water stops used more recently at TVA's Hiwassee Dam overcome many of the objections to those used at Norris.

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*TVA staff member.

Chapter 7

RESERVOIR ACTIVITIES

Norris Reservoir, with a 705-mile shore line at elevation 1,020, extends into five counties. The creation of the reservoir necessitated considerable diversified work such as surveying and mapping and purchasing nearly 150,000 acres of land, moving 3,000 families and 5,000 graves, clearing forest and structures from the draw-down area, relocating affected highways, railroads, and utility lines, and providing adjustments for other backwater damages. Labor from Civilian Conservation Corps camps stationed in the valley was not utilized for any of this work which was strictly necessary for the completion of the project to serve purposes of navigation, flood control, and power. However, such labor was used for related work in the fields of erosion control, park development, recreation, and general beautification of the area.

SURVEYS AND MAPPING

The use of aerial photographs for property surveys and mapping in the Norris Reservoir was a noteworthy development. There was only little experience as a guidance in surveying by this method some 187,000 acres of the steeply sloping area. These property surveys, accurate for acreage to within 1 percent (average), were the basis of the land acquisition program with which approximately 150,000 acres of land were acquired with surprisingly few litigations. In addition to these property surveys, there were, of course, other surveys common to large reservoirs that were conducted in the conventional surveying manner.

On May 1, 1933, a short time before the establishment of the Tennessee Valley Authority, the United States Army Engineers district office at Chattanooga concentrated a large surveying force in the Cove Creek (Norris) Reservoir area with offices at La Follette, Tenn., and plans were laid for surveying work. Special consultants were secured from the Army Engineers district office in the upper Mississippi Valley region to advise on procedures and to expedite the work. Most of this original force was later transferred to the Authority.

Preliminary surveys.

The original plan adopted by the United States Army Engineers provided for a temporary control survey, the marking of the 1,060 contour originally considered to be the "taking line," and the preparation of land acquisition surveys of each tract to be purchased. The control surveys were of third order accuracy with the traverses following the principal river valleys, and with the levels being based on the United States Geological Survey bench marks. No permanent bench marks of this survey were established as it was known that they would be later covered with water.

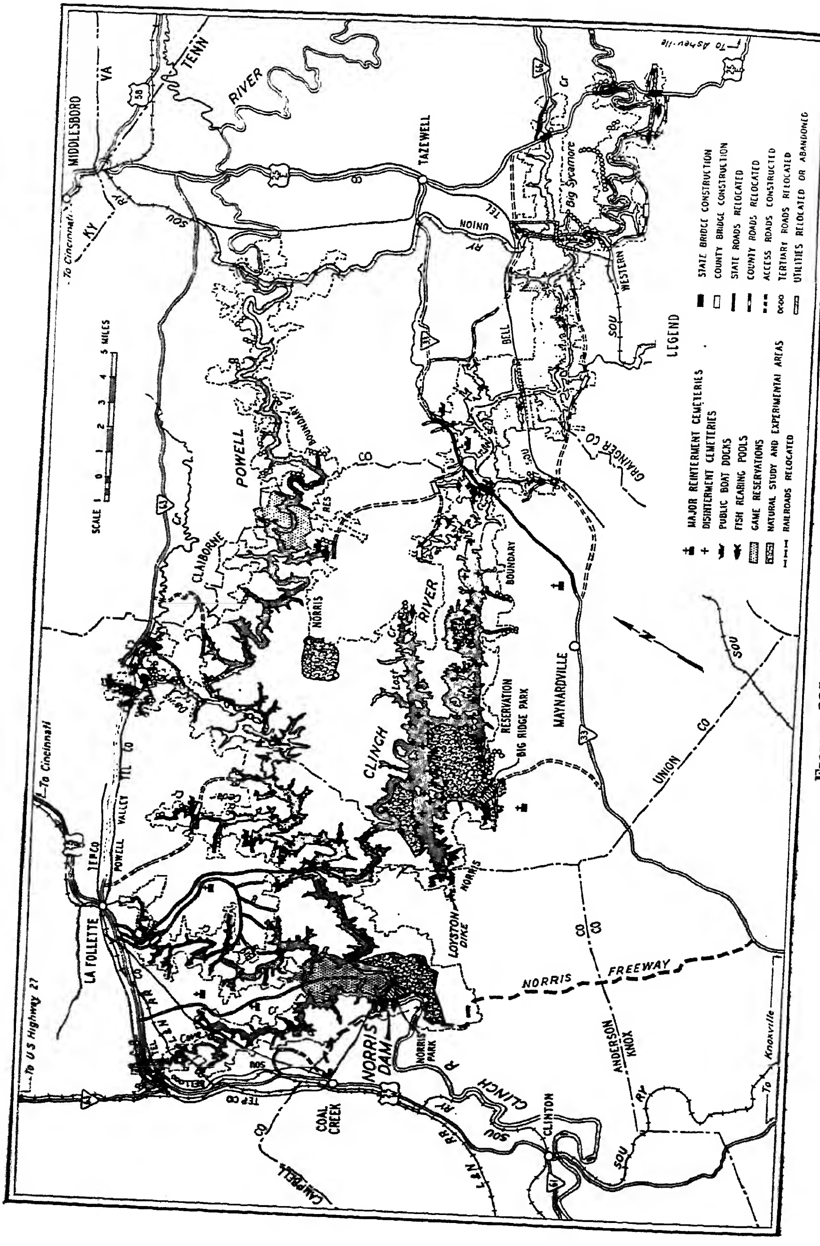

FIGURE 225.—*Reservoir activities.*

Property surveys were first made from transit-tape surveys, being tied to the temporary control traverses.

Final survey plans.

Field work was carried on under the United States Army Engineers' preliminary plan until the work was taken over by the Authority on August 1, 1933. The temporary control surveys were abandoned, the permanent control being combined with a series of silt range surveys.

The schedule for building the dam was such that it became apparent that too much time would be required for property surveys being made by the original method. Also, the cost seemed unusually high in comparison with land values. Accordingly, experiments made with aerial photographs for mapping led to abandoning the original plan and adopting a new plan based almost entirely on aerial photographs.

The regularly scheduled surveys[1] discussed in this chapter may be summarized as follows:

Basic control	466 miles; 3,751 monuments.
Aerial photographs	511 miles traverse; 522 miles levels.
Land acquisition—Property surveyed	905 square miles.
Reservoir clearing—1,020 and 960 contours marked.	186,700 acres. 1,064 miles.
Boundary line	466 miles; 3,751 monuments.
Silt ranges	516 ranges; 108 miles; 988 monuments
Cemetery surveys	326 cemeteries; 12,005 graves.
Utility relocations	24 miles of line.
Large scale topography—Principal surveys.	13,200 acres.

Basic control.

In addition to the previously mentioned temporary basic control, a permanent basic control was combined with silt ranges spaced approximately ½ mile apart, each range extending entirely across the reservoir. There were established 516 such ranges, each end of which was permanently marked with a concrete monument (988 monuments, some being common to two or more ranges) set just above the 1,020 shore line. Specifications for the permanent control were more rigid than would have been necessary for silt ranges and called for position accuracy of 1 part in 5,000, all monuments being referenced and computed to a plane coordinate system. Elevations were of third-order accuracy and were on the 1936 Supplementary Adjustment mean sea-level datum.

Plane coordinate system.—The Tennessee plane coordinate system established by the United States Coast and Geodetic Survey in 1935 was adopted as the computing and plotting base for all reservoir surveys and maps except the original temporary traverse and level surveys.

Level datum.—The third-order levels started by the Army Engineers were continued by the Authority. They were based on United States Geological Survey bench marks in the vicinity which were on the national level datum, known as the 1912 Adjustment. This datum was used for all construction features. The silt range basic control surveys were, however, based on the later national level datum—the

[1] Sayford, Ned H., Surveying and Mapping in the Tennessee Valley, Civil Engineering, December 1935.

1936 Supplementary Adjustment Datum. The difference at Norris Dam between the two datums is 0.3 foot, the 1936 elevations being higher. The elevation differences throughout the reservoir area vary from about plus 1 foot to minus 1 foot, these differences being caused principally by accumulated errors in the old level lines.

Temporary traverse and levels.—Since it was known that this temporary control would be inundated and would be replaced by a permanent control system, no attempt was made to make final adjustments, convert to the State coordinate system, or to publish lists of station and bench mark descriptions, positions, and elevations.

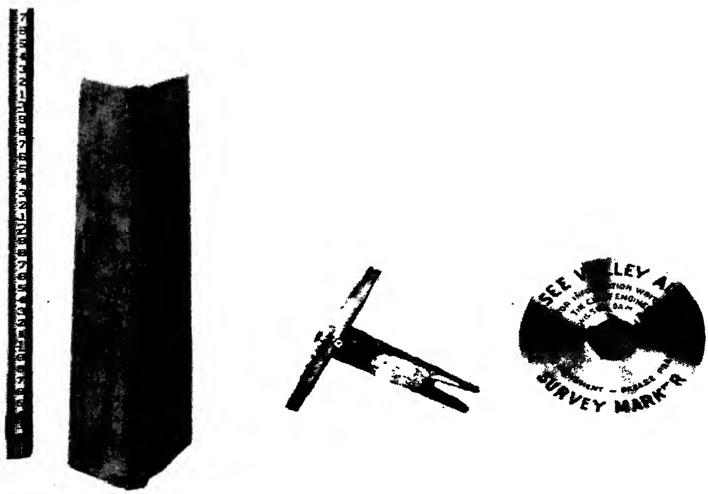

FIGURE 226.—*Silt range and control survey marker.*

Silt range surveys.

These ranges were established for the purpose of measuring the depth of silt deposits on the reservoir bottom in the future by preparing and comparing profiles of each range. The work fell into four operations: locating and setting monuments at specified sites, leveling to establish third-order elevations, taping and profiling ranges, and establishing plane coordinates of monuments.[2]

Monuments.—The silt range monuments used were precast and made of reinforced concrete. In the top of each monument is embedded a bronze tablet bearing identification inscription as shown in figure 226.

Levels.—Third-order levels were run along the 1,020 contour line on both banks of each stream using the monuments as turning points. Where possible this line was tied from both banks to temporary bench marks in the lower valley. Thus, in effect, a triple run level line was run along each stream in the reservoir.

[2] Whitmore, George D., Control Surveys in Norris Reservoir, The Military Engineer, July 1935.

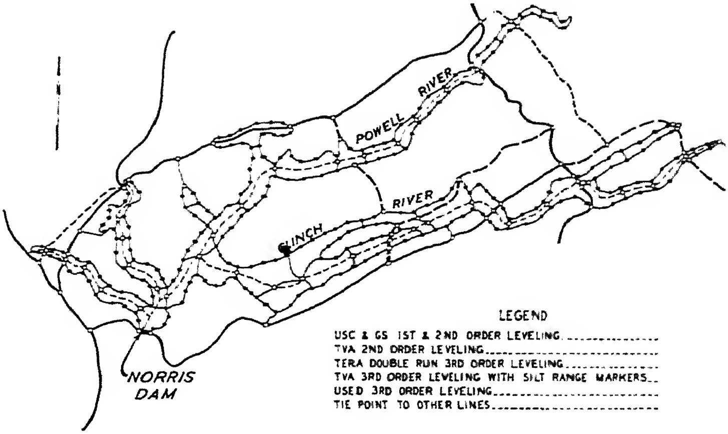

FIGURE 227.—*Arrangement of level control lines.*

Profiling and taping.—The taping of each range and the profiling, including river soundings, were done simultaneously using methods similar to standard mine-surveying practice of slope taping vertical-angle profiling. Using a standard 500-foot tape, the procedure was

FIGURE 228.—*Slope taping procedure.*

to make slope measurements from the center of the transit to a hub, then from the hub to the transit, and so on, using a length of tape up to 500 feet that would swing clear of the ground.

The 500-foot tapes were calibrated[3] with a master tape which had been checked with the United States Bureau of Standards, and with

[3] Whitmore, George D., Control Surveys in Norris Reservoir, The Military Engineer, July 1935.

each tape was provided a temperature and tension table to determine the proper tension to use for each chosen length. To secure a check on taping, two independent distances along each range were taped simultaneously. This was accomplished by setting two hubs about 10 feet apart for each transit set up. These two independent measurements for each range were required to check within 1 part in 10,000 for distance, and within 1 foot for total difference of monument elevations.

Similar profile shots at approximately 50-foot intervals and at all breaks in grade were taken at each transit station. These were faster and slightly less accurate. At breaks a shot consisted of a taped slope distance and vertical transit angles.

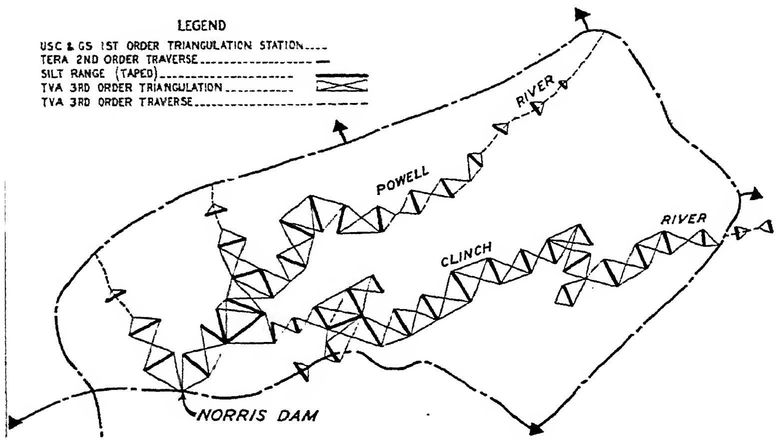

FIGURE 229.—*Typical triangulation figures and traverse schemes used to coordinate silt range and control survey monuments.*

Plane coordinates.—Connections between silt ranges were made where possible by third-order triangulation methods, using the silt ranges as measured base lines. Where triangulation could not be used, third-order traverse connections were run. Triangulation or traverse connections were made to the second-order traverse monuments established by the Tennessee Emergency Relief Administration under the direction of the United States Coast and Geodetic Survey.

After the triangulation figures were closed, the chain of figures was treated as a traverse line, selecting a route which included every silt range as a course of the traverse. The plane coordinate position of each monument and all junction points within the reservoir were determined. The average adjustment between junction points was well within specifications of 1 part in 5,000.

Filing of control monument data.—The approximate location of each monument is shown on a control index sheet (fig. 230) and a complete description of each silt range monument is shown on a 5- by 8-inch card (fig. 231). These index maps are photographic reductions from planimetric maps reduced to a scale of 1 inch equal to 1 mile compiled in 15-minute quadrangles. The record cards and index maps are permanently filed with TVA record drawings.

Aerial photographs.

Aerial photographs covering 905 square miles of the Norris Reservoir were taken with a single-lens camera of about 8½-inch focal length from a height of about 10,000 feet with a resulting contact print scale of approximately 1:15,000 (1 inch equals 1,250 feet). The contract for this work was awarded to the Tobin Aerial Co., San

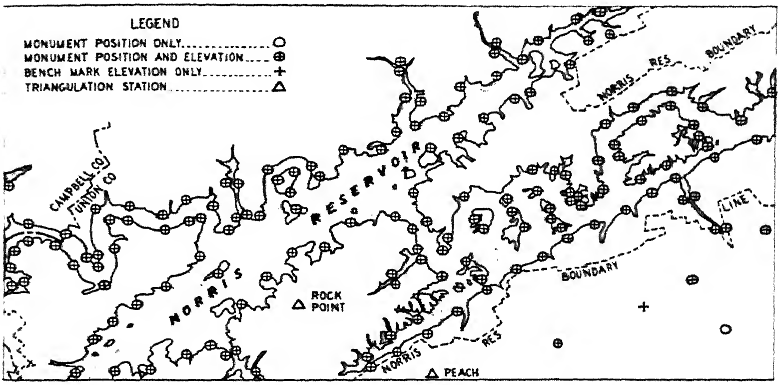

FIGURE 230.—*Typical section of control-index sheet.*

Antonio, Texas. Controlled mosaic sheets in 3.75-minute quadrangles and enlargements or reductions were later provided for ratios designated by the Authority.

To determine the scale for aerial photographs, distances were measured between identifiable points on every fourth or fifth picture on every strip of photographs and the scales for intermediate ones were interpolated. Elevations were determined approximately by aneroid barometers so that compensation could be made for relief displacements. Scales were thus determined for each original aerial photograph negative for a datum plane of elevation 1,000. On ratioed photographs, areas higher than elevation 1,000 would be to a larger scale, while those below would be at a smaller one.

Practically no literature was available on this subject when this work started, so the field and office procedures had to be developed [4] as the job progressed.

[4] Tubis, Harry, TVA Photogrammetrist, article in News Notes of the American Society of Photogrammetry, July–August 1935. Anderson, R. O., Applied Photogrammetry, Edwards Brothers, Ann Arbor, Mich., 1937.

Land acquisition surveys.

Surveys for land acquisition were only sufficiently complete and accurate to determine the correct acreages and to prepare legal descriptions of lands to be purchased. As most of the acquired area

MONUMENT DESCRIPTION

STATION CR 200-R Date Set 1935 Norris Res. SR Survey

State Tennessee County Grainger Section Quad Rutledge-154

Character of Station Standard TVA bronze tablet set in precast concrete monument and stamped "CR 200-R-1935"

Distance and Direction from Prominent Points
Station is located 2.2 miles Southeast from Days Mill Church, 3.2 miles Southeast from Beech Grove Church, and 1.0 mile South from Shelton Bluff.

DETAILED DESCRIPTION OF LOCATION
Road or Highway 1.8 miles Northwest from Puncheon Valley Road

Property Owner

Other Descriptions Monument is 1.6 miles East from Railroad Station Dutch, 2300' Northwest from Puncheon Camp Creek and 1.3 miles Northeast of Salem Church. It is 600' below mouth of Cracker Creek, on steep slope in woods, 40' from outstanding 24" Beech tree and 15' from 1020 contour.

REFERENCE MARKS
Mark No. 1 61.8' S 76 00 E to 24" Beech Tree
Mark No. 2 44.2' N 60 00 W to 12" Oak tree
Mark No. 3 23.2' S 53 00 E to corner fence post

FRONT OF CARD

TVA 1160 (9-1-37)

STATION CR 200-R
AZIMUTH MARKS
Mark No. 1 Silt Range Monument CR 200-L

Distance 420.3 Mag. Bearing S 42° 30' W
Grid Azimuth from Station to Mark No. 1 42° 29' 18"

Mark No. 2

Distance Mag. Bearing
Grid Azimuth from Station to Mark No. 2 ° ' "

Elevation: Adjustment 1936 Southeastern 1025.6
Position
Grid Tennessee Lambert North (Y) 723,020.4 East (X) 2,728,911.3
Geographic Latitude 36° 41' 23".45 Longitude 83° 22' 45".56

Distance from To

Remarks and References:
Volume 16, page 23

BACK OF CARD

Figure 231.—*Specimen data card of silt range on control survey monument.*

would be flooded, these surveys were not necessarily accurate enough to serve as a basis for later retracement surveys.

Property reconnaissance.—Before the actual property surveys could be started, a property ownership index map was made, as none were available from the various counties. Workers obtained for each property the owner's name, the date of purchase or inheritance, ap-

proximate boundary lines, and other ownership data so that deed copying and the later property surveys would be facilitated. Boundary lines were sketched on contact prints of aerial photographs and later transferred to the 1:15,000 scale United States Army Engineer topographic sheets from which reconnaissance map sheets were traced in pencil. These maps, covering 214,800 acres, served the purpose of general plan and progress maps for property surveys and as field maps for preliminary appraisals. Later they proved valuable to the field parties making property surveys.

FIGURE 232.—*Enlarged aerial photograph used as plane table sheet.*

Deed copying.—Copies of each recorded deed were made from county records. Deeds were secured from the owner when none was recorded. In each major discrepancy between deed description and field reconnaissance sketches, both were checked before actual property surveys were made. A total of 4,114 deeds were copied.

Property survey methods.—Farm tracts varying in size from about 10 to several hundred acres were surveyed almost entirely by the use of aerial photographs. All small tracts of less than 10 acres and a few less than 20 acres were made by plane table, using larger scales than those used on photographs. Transit tape methods were used for city and subdivision lots where record plats of such lots were not already available.

Aerial photographic property surveys.—Enlargements of aerial photographs to the scale of 1 inch equal to 500 feet were the basis for farm surveys. Ninety-five percent of these enlargements had a scaling accuracy to within ½ of 1 percent and only occasionally did the error exceed 1 percent. The "net center area" (that area defined by mid-overlap lines) of each print was marked and the field surveyors plotted as much as possible within this center area so as to minimize the effect of relief displacement and tilt.

In addition to the photographic enlargements, the surveyors were furnished with property reconnaissance maps, copies of deeds, and record plats where available.

The procedure then consisted of locating on the ground and plotting on the photograph each property corner. If the corner was

TVA 834
Tennessee Valley Authority

Sheet 1 of 1 Sheets

NORRIS RESERVOIR LAND SURVEY

Owner's Name Mrs Doll Helton Tract No. 1862 Photo Nos. 450-3-96

Section Nos. — Twp. — Rge. —

County Claiborne Civil Dist. 3 State Tennessee P.T.S. Nos. —

Location Right bank Clinch River, above Whistle Branch - River Mi. 54

Date June 13, 1934

Party Chief George M. Pace

Rec. Joel A. Hightower

Rodmen F. Whitehead

Deed Nos. 324 Deed Book No. 46-3 Page 62

Weather Fair

INVENTORY

No.	Type	
1	Dwellings	1 story frame
—	Barns	—
—	Stables	—
2	Sheds	1-1L, 1-1F
1	Smokehouses	1F
	Wells or cisterns	
—	Garages	—
1	Cribs	1F

Other improvements (List any dams, mines, quarries, gravel pits, filling stations, water systems, sewage systems, tile drainage, cellars, well-houses, stores, or other items) None

Springs: How many are used to supply dwellings?

Roads: Remarks on 14' County through tract

Number of Orchards 1-¼ Acre Apple & Peach

(Symbols: One story frame building, 1F; Two story brick building, 2B; Stone, S; Corrugated Iron, Cor.I; Log, L.)

Comments on deed discrepancies

Bridges

No.	Type	Span	Width
None			

Power & Phone Lines

Type Pole	No. Wires	Voltage	Phase	Owner
	None			

Cemeteries

Name	No. Graves	Lowest Elev.	Public or Private
None			

Show all features noted above on photographs and/or plane table sheets.

Checked by D.L.K. Date June 21, 1934

Note: Use reverse side of this form if additional space is required for survey notes or comments on deed discrepancies.

FIGURE 233.—*Typical property inventory form.*

directly identifiable on the photograph, the plotting was merely a matter of precisely pricking the proper image. If the corner was not directly identifiable, then it had to be established on the photograph by conventional plane table traverse methods, utilizing the photograph itself as the plane table sheet. This was accomplished by originating

and closing the traverse on identifiable image points selected as near as possible to the corner to be plotted. Relief caused a displacement of each image toward or away from the center of the photograph,

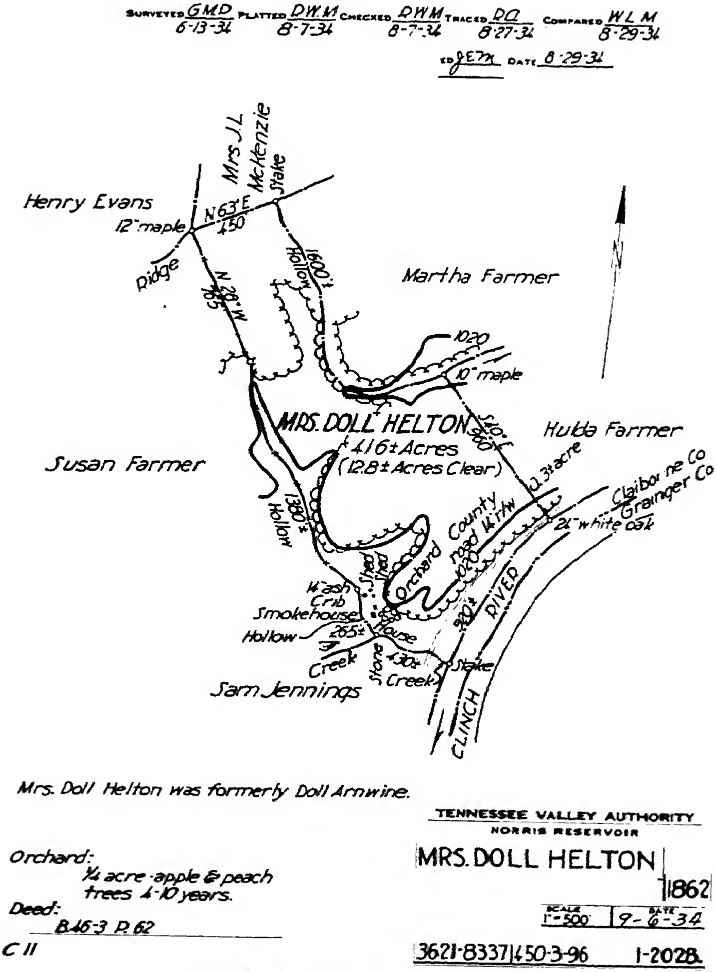

FIGURE 234.—*Typical property plat.*

making necessary a correction which had to be applied in the field before the plane table traverse could be closed and adjusted. This correction was based on the elevation of the point and the distance from the center of the photograph. These elevations were established

within about 25 feet by means of barometers. Barometric readings were also taken as the identifiable corners as the basis for later correcting for the relief displacement of these points in the office.

When owners could not definitely point out their corners, conventional transit-tape surveys were made from deeds and other evidence to establish the corners on the land before plotting them on the photograph.

The surveyors delineated in pencil on the photograph all of the structures and features which were to be shown on the final plat and made necessary descriptive notes of each property corner and property line. For assistance to the draftsmen, an "inventory" of structures and improvements on each property was made as shown in figure 233.

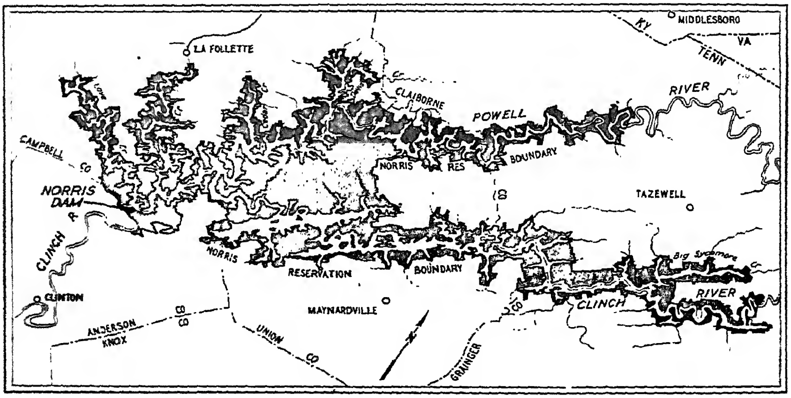

FIGURE 235.—*Land purchased for reservoir purposes.*

The 1,020 contour which had been marked previously on the ground was plotted on each photograph in correct photographic location. Also, the 1,047 contour (approximate minimum buying line) and the 940 contour (lower clearing limit) were mapped. For many tracts requiring special consideration, 5-foot and even 2-foot contours were plotted.

Office drafting of plats.—The first operation in the office was to trace from the photograph on vellum the photographic position of each property corner. Correction was then made for relief displacement in order to establish the correct map position. Reference tables were provided for this purpose which eliminated computations. As tilt was a serious factor in only a very few cases, the photographs were normally assumed to be tilt free. Bearings and distances of each property line were then scaled from the tracing and acreage in each tract was planimetered. The acreage of cleared land and wooded land to be cleared was also planimetered.

Complete individual plats for each tract were then traced from the vellum overlay. To these were added bearings and distances of boundary courses, the acreage figures, and all necessary notes. Plats

of small tracts were traced directly from the white paper plane table sheets.

The property "taking line" was then determined, as described later in this chapter, and plotted on each plat with the distance and bearing of any severance courses. A "metes and bounds" description of each tract to be acquired was then prepared and served as a basis for land acquisition.

Area surveyed.—Land acquisition surveys were made for several different purposes. Those most commonly used were:

	Number tracts	Acres
Reservoir land surveys	2,821	137,000
Peninsular land surveys	618	29,000
Norris town site surveys	30	1,000
Submarginal surveys	188	14,000

FIGURE 236.—*Marking contour in wooded area.*

All tracts in the downstream portion of the reservoir touched by the 1,047 contour were surveyed. In the flatter upstream portions, surveys were stopped at approximately the 1,030 contour, and in the head of the reservoir, especially on the Clinch and Powell Rivers, they were terminated where the 1,020 contour crossed the water surface.

Reservoir clearing surveys.

Fourth-order levels were used in tracing 705 miles of the 1,020 contour line on the ground. In wooded areas marks were painted in white on trees, selecting trees of such elevation that the marks would fall about 1 foot off the ground. In cleared areas the line was marked by white stakes and on fence posts and structures spaced

at intervals of from 50 to 200 feet. Levels were tied to control bench marks about every 2 to 6 miles with an accumulated error seldom exceeding one-half foot. The 940 contour totaling 359 miles and marking the lower clearing level was marked with red paint in a similar manner.

White marks were also placed around all sink holes within the area whose bottom fell below elevation 1,020. On hazardous bluffs the marks would be extended from each side as far as possible. Inspection indicated that 95 percent of the contours were correct to within a few tenths of a foot.

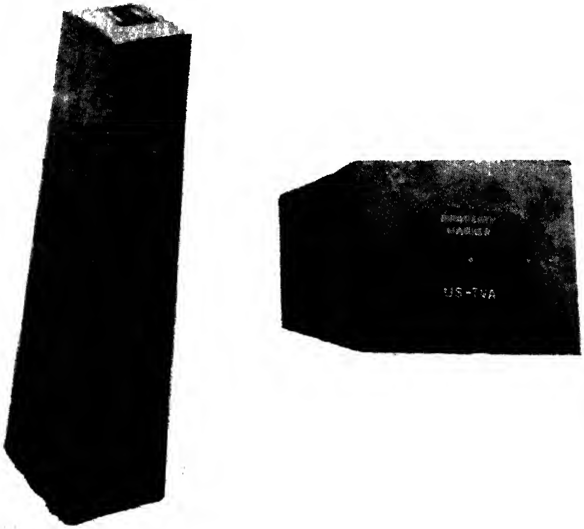

FIGURE 237.—*Property marker monument.*

Reservoir boundary line survey.

A total of 3,751 precast concrete monuments were set, 1 at each angle point in the outside boundary line of the reservoir reservation. Where a natural marker already existed, a TVA monument was placed as near the corner on TVA land as possible to serve as a reference for the natural corner. Where a straight course of the boundary intercepted a meandering course such as a stream, a monument was set on line on the straight course as near as possible to its intersection of the meandering course.

After all monuments in a section had been placed, fourth-order transit traverses totaling 466 miles for the reservoir were run to connect the monuments. Where practicable, the monuments were used as turning points in the traverse. Side shots were made from stations on the traverse to all other monuments and natural corners.

At intervals of approximately 5 miles, the fourth-order traverse was connected to the basic control which in most cases was made up of the silt range monuments. From these connections the coordinate position of each boundary corner was computed. Position closures averaged about 1:2,500 with none greater than 1:1,000. The length and bearing of each boundary course was then computed from the adjusted positions of the corners. It was found that the difference between these computed values and the corresponding values scaled from the photographs was always relatively small.

The boundary line survey was reduced in final form to the Tennessee plane coordinate system and a record plat is being prepared using planimetric maps to the scale of 1:24,000 as the base sheets. These sheets will show each angle point and its index number, except that in the case of many points close together some corners will of necessity be omitted from the plat. On a tabulation sheet attached to each map will be shown statistical data concerning each point.

Cemetery surveys.

A survey and plat on a 1:240 scale was prepared by plane table for each disinterment and reinterment cemetery. Much difficulty was experienced in identifying graves as markers in many cases had disappeared, especially in the case of the older graves, many of which dated back more than 100 years. A total of approximately 12,000 graves, located in 232 private and church cemeteries, was surveyed. A total of 5,226 of these was relocated and surveyed in 94 reinterment cemeteries. The total cost for cemetery surveys and maps was $35,535.38, exclusive of general expense.

Communication and power line surveys.

Special surveys were made of all public utilities such as power transmission and communication lines within the area to be flooded. The location of these lines was profiled and mapped in detail and inventories were made of all structural items. Data regarding past operations and the number of customers were frequently necessary. A total of 24 miles of such surveys was made.

Topographic surveys.

Since the United States Army Engineers had completed the Tennessee River survey topographic sheets for this area (scale 1:15,000), a general topographic survey was not made by the Authority. There were many uses, however, for large scale topographic surveys and maps, the principal ones being:

	Scale	Contour interval	Area mapped
		Feet	*Acres*
Norris Dam site	1 inch=50 feet	2	825
Dam site	1 inch=200 feet	5	4,350
Norris town site	1 inch=200 feet	5	5,940
Highway relocation (five State highway projects)	1 inch=50 feet	2	865
Loyston Divide	1 inch=200 feet	2	765
Town of Caryville	1 inch=50 feet	2	210
Eagle Bend Tree Nursery	1 inch=200 feet	2	135
Proposed Norris National Cemetery	1 inch=100 feet	2	45
Proposed Norris Recreation Park	1 inch=50 feet	2	20

These were made by plane table and where possible were tied with transit tape traverses to coordinate control points. All well-defined cultural features were plotted correctly on the finished maps within 0.02-inch of the true coordinate position. Ninety percent of the elevations interpolated from the maps were correct within one-half the contour interval.

LAND PURCHASE CONTROL

It was stipulated that requests for land purchases for any purpose should be in writing and should be reviewed and approved by one central organization, in order to establish a control for land purchases. This organization alone was authorized to approve purchases, within limits prescribed by the Board of Directors, and to prepare definite descriptions and maps for land acquisition.

It was agreed that the Authority would acquire every tract of land lying within one-quarter mile of the water line at design flood stage, elevation 1,047. The most fertile and productive land was in the valleys with only small hillside portions extending beyond the one-quarter mile limit. As these portions in most cases would be isolated and worthless, it was also agreed to acquire those containing not more than 40 acres when the owner desired to sell. This was later interpreted to be the limit of purchase rather than the extent to which it was necessary to acquire land.

The acquisition of lands immediately beyond the flowage line was justified by a combination of several reasons. To have bought only to the flowage line would have caused payment of large severance damages which would have about equaled the cost of taking additional lands. This was particularly true in the cases where the productive farm lands were largely within the flowage area. It also permitted a larger degree of local erosion control to reduce reservoir silting and served as a protection against possible damage claims resulting from changed ground water conditions.

Establishing "taking line."

It became necessary to make careful studies of the many miles of shore line to apply this general purchase authority. Each tract which appeared likely to be affected by the reservoir was studied. Property surveys made mostly from aerial maps were used in connection with the aerial photographs viewed under a magnifying stereoscope. Usually these maps and photographs together with additional information furnished by surveyors were sufficient for determining the purchase boundary. The many other factors considered included:

Cost of roads to serve isolated tracts.
Effect on social and economic conditions.
Soil erosion conditions.
Possible recreational and scenic value.
Pool levels and flood frequencies.
Land values versus severance damages.
Levee protection possibilities.
Mosquito control problems.
Nature, extent, and probable value of mineral deposits.
Authority's legal liability.

Land purchases under the general authority.

In a small number of cases, perhaps 5 percent, it was found that the desired land could be acquired and that the remaining property would be capable of continuing as a farm unit as in the past with perhaps a small expenditure on road replacements. Where such conditions existed, severance lines were placed on the property and only a portion of the tract was acquired.

By far the largest portion of the land acquired fell within the general purchase authority. The most serious land purchase problem was the flooding of roads and the almost prohibitive cost of their replacement, especially for small areas. For those isolated properties which could be bought more cheaply than roads to them could be built and which were beyond the Authority's authorized buying limit, special action by the Board of Directors was requested.

In most of the reservoir the 1,047 contour was included within the land purchases. Yet, because of probable infrequent flooding, it appeared advisable to stop land purchases short of where this contour closed along flat tributaries and in the upper reaches of the reservoir. In these sections small parts of farms extending below the 1,034 contour (top of gates) were not acquired if their purchase would have caused large severance damages. Approximately 274 acres of such lands below elevation 1,035 were not acquired. This land is distributed among 137 tracts—an average of about 2 acres per tract. In addition to being proportionately expensive, the acquisition of these parcels of land would have reduced the already small amount of cultivable land and would have in many cases divided a farm unit. Further study in connection with actual operating conditions, however, may reveal the advisability of small additional purchases in these lower areas.

Purchases requiring special Board action.

As previously mentioned, it was cheaper to buy certain isolated properties outside the buying limits than to build new access facilities for them. In a few cases, however, where the cost of acquiring the land was slightly more than the cost of providing access, the land was purchased because of other considerations. In each such case it was necessary to prepare brief reports with sketch maps for Board approval. The most outstanding of this type was the peninsula between the Powell and Clinch Rivers where some 14,300 acres were acquired in lieu of bridge and road construction.

Purchase of lands for other definite purposes required separate Board resolution. Included in this group were lands for experimental work, the town of Norris, the Norris town water supply, watershed, freeway right-of-way, and the Coal Creek storage yards.

Total land purchase approvals.

The following tabulation shows the division of land purchase approvals. All of the first item, "Purchased under general authority," is reservoir land, while the last three items were approved for other uses as indicated:

	Percent
Purchased under general authority	73.9
Small isolated peninsulas and single tracts	6.2
Central peninsula area	9.4
Lands for special uses	10.5
Total	100.0

The following table showing the acres of land involved by the Norris project indicates that only 19,840 acres of cleared land, about 13 percent of the total land purchased, will be submerged. Not all of this was used for farming purposes.

	Acres
Pool area at elevation 1,020	34, 200
Area between original river banks	2, 913
Area submerged outside original river banks	31, 287
Wooded area prior to TVA clearance work	11, 447
Cleared area prior to TVA clearance work	19, 840

LAND ACQUISITION AND EASEMENTS

Approximately 152,000 acres of land were purchased in connection with the Norris project. Of this property, 94.17 percent was acquired by voluntary transfer; 1.46 percent had to be condemned for title reasons; and 4.37 percent had to be condemned because of refusal to sell at the appraised values.

Organization.

The land acquisition organization was divided into three natural sections—appraisal, buying, and title. These sections were responsible for the acquisition of all interests in real estate required by the Authority.

Appraisal section.—A careful study was made by the TVA board of appraisal and review of each area to be acquired, and classifications and evaluations of the various types of soils and improvements were established. With the previously prepared property map, appraisers went over the properties with the owners, preparing detailed appraisal reports. These reports were carefully reviewed by the chief appraiser and the board of appraisal and review, which fixed the final price to be offered for the property. The prices fixed by the appraisal section were turned over to the buying section for negotiation.

Buying section.—Negotiations were made by the buying section with the owner for a contract for purchase and sale of the property at the price fixed by the board of appraisal and review. No price-trading was permitted to enter into the negotiations, and the property could not be purchased above or below the figure set by the board of appraisal and review. All efforts were directed towards convincing the owners of the fairness of the price offered as a result of an impersonal, impartial appraisal by a competent appraisal staff, familiar with local property values.

Title section.—While the appraisal work was proceeding, abstracts of titles were made under the supervision of title closers. Contracts for each tract were turned over to the title section; and a title closer, after clearing all defects in title, would report in regard thereto and, if favorable, would request the issuance of a check for payment of the purchase price. Upon receipt by the title closer of the check for payment, the transaction was closed and the deed for the property recorded. Transactions were usually closed within 30 days after execution of the contract.

Acquisition policies.

The appraisal procedure was designed to fix a uniform price for elements of value in the same class. The standards of measure fixed

were such that the lands were acquired at prices which would enable the owners to relocate and re-establish themselves in situations which would afford them contentment equal to that which they previously enjoyed.

Landowners were permitted to remove such improvements as were not needed by the Authority, the appraisal taking into consideration such salvage value. Owners were also permitted to remain on the land until possession was needed by the Authority. Surrender of possession was fixed during the winter season so that there would be no liability for growing crops.

Although it was the policy of the Authority to acquire the unencumbered fee of the property needed, an exception was made where there were outstanding mineral rights in properties lying above the flood level of the reservoir. No attempt was made to acquire such mineral interests, which, of course, resulted in certain economy to the acquisition program as well as avoided retarding any mineral development in the area.

The non-price-trading policy was not applied to the acquisition of easements, as the small considerations involved in acquiring such rights did not warrant the expense of incorporating this type of acquisition under the appraisal procedure.

Condemnation proceedings.

For titles which were not entirely clear but which could be made clear by filing a bill in the State courts, this method was used. However, in certain instances, titles were in such condition that they could not be cleared through any ordinary procedure, in which event the properties were condemned. When it was finally determined that a property could not be acquired by voluntary transfer, it was referred to the Authority's legal staff for condemnation, the land acquisition organization being responsible for providing the evidence of value, both through members of its own staff and through outside disinterested parties.

TABLE 92.—*Land purchases in fee and land easement purchases for Norris project as of Sept. 30, 1938*

	Tracts	Acres	Land cost	Improvements	Total cost	Average cost per acre
Distribution of land purchases in fee:						
Reservoir	2,734	141,770.06	$4,684,436.03	$2,960,975.51	$7,645,411.54	$53.93
Norris freeway	53	272.28	16,248.11	2,705.00	18,953.11	69.61
Distribution of land easement purchases:						
Highways	234	230.084	18,888.98	16,955.95	35,844.93	155.79
Flowage	5	36.18	2,860.00		2,860.00	79.05
Total	3,026	142,308.604	4,722,433.12	2,980,636.46	7,703,069.58	54.13

NOTE.—The above totals exclude 32 tracts in process of condemnation and 145 tracts in and around the town of Norris.

Land acquired.

An analysis of the total land acquired for the Norris project is shown in table 92. As of September 30, 1938, a total of 2,787 tracts containing 142,042.34 acres was acquired at a cost of $7,664,364.65

including improvements; and a total of 239 tracts or rights was acquired for the highways and flowage easements containing 266.264 acres at a cost of $38,704.93 including improvements. The average reservoir tract was 50.5 acres and cost $31.51 per acre for land and $20.17 for improvements. These prices are slightly higher per acre for land and improvements than were paid in the lower main river reservoirs. Although land for the main river projects was about 30 percent more expensive than for Norris, the improvements were only about $5 per acre as compared to $20 for Norris. The greater density of population in the Norris Reservoir resulted in the higher improvement value; also, the tracts were only about half as large and the percentage of resident owners was much higher.

The total land acquisition cost for the land and rights shown in table 92, including surveying, mapping, land purchase control, appraising, buying, condemnation proceedings, and title closing, through June 30, 1938, amounted to $1,037,170.54.

PREPARATION OF RESERVOIR FOR FLOODING

The preparation of the reservoir for flooding presented unique and difficult problems because of the mountainous region and the personalities involved. Much planning was necessary, and legal basis for making policies had to be established. This was particularly true with respect to family removal and cemetery relocation. However, this work progressed in an orderly manner with all work being completed on schedule. Close cooperation necessarily existed between the agencies performing the work, and at times an exchange of duties was made.

FAMILY REMOVAL

Only a very little established precedent was available for guiding the program of removing families and personal property from purchased land. The TVA program was based largely upon the results of the questionnaire made late in 1934 and discussed in some detail in chapter 2. In many instances persons selling lands vacated them as provided in the Authority's contract—free of all personal property. As was expected with 2,899 rural families in the purchased area, most of whom had lived in the same locality for generations, there appeared to them insurmountable difficulties in finding suitable relocation sites. Many families moved to temporary homes pending settlements of land claims, not knowing what financial resources would be available for permanent homes. Owners had but little knowledge of available sites, and it was difficult for them to find desirable land within their ability to purchase. Also, some were antagonistic toward the Authority's program, although very few refused to cooperate in any removal plan.

Many strange and complicated problems were presented. One property owner insisted that the Authority agree to allow his pet frog to remain in a spring where it had lived since its tadpole days. Some of the largest property owners felt that the Government was forcing them out of their rightful homes for a new civilization in which they had little interest. Like their pioneer forefathers who chopped their way through heavy forests to settle this country, a small group of

them considered the possibility of leaving the United States and establishing a new colony in Brazil. Efforts to persuade some of the families to relocate in lower and more productive land or in other sections met with little success—the mountain families did not wish to become "lowlanders." A great majority preferred to find homes in similar surrounding regions.

Out of the 2,899 families living on lands purchased in the reservoir area, tenant families constituted about 41 percent. These tenant families presented the greatest relocation problem, since they received no payment for land and, as a result, to them moving was especially difficult because of meager financial resources. With over 140,000 acres of land taken out of cultivation, they were forced either to move long distances or to relocate into areas where productive lands were already somewhat overpopulated.

The cost to a given family for transporting household goods to a new location ranged from about $10 to $300 or more and averaged about $75. The median family paid about $50, and it should be noted that the reputed per capita annual spendable income for this area in 1935 was but $90.

Such problems as these were recognized and provision made for solving them by the TVA Act as amended in 1935.

Family relocation service

A relocation service was established [5] in cooperation with the University of Tennessee Agricultural Extension Division to assist the affected families in locating and contracting for new homes. However, there was a number of families who could not be served by this advisory assistance, chiefly because of their economic poverty or uncooperative attitude. The progress of the dam construction and danger of floodwaters necessitated more specialized removal service. Consequently, the Authority established a supplementary family removal service in September 1935 to deal with special problems and to cooperate with all interested agencies in the removal and relocation of the 1,016 families who still remained in the purchase areas.

The TVA staff consisted of 10 to 15 social case workers who were, in general, persons having previous social work experience and who were familiar with local conditions. This organization functioned chiefly as a service agency to establish the proper contacts between individuals and resources. This included referring the families to agencies likely to give aid and assistance, encouraging agencies to assist in the readjustment of families, and giving advice to families in making use of their own resources. The relationship of individual families to the different sources of possible assistance is shown in figure 238.

Emergency removal preparation.

After the closure of the dam had been made, in the fall of 1935, water could be released only through the eight discharge conduits. Even with all available discharge outlets open, a heavy rainfall would have caused a rise of many feet within a few hours, especially in the lower part of the reservoir. Some of the families who had

[5] Authorized in sec. 4–L of the TVA Act as amended in 1935. See Appendix J.

not moved were therefore in danger of being flooded in the event of heavy rains, and emergency removal preparations had to be made.

One cruiser and several boats were kept in readiness at the dam. Emergency equipment such as tents, stores, cots, and camping equipment was secured through the United States Army and stored at the dam. A survey of the families living below elevation 940 was made, and the most feasible emergency routes to these homes were determined. Arrangements were made with C. C. C. camps near the reservoir to feed families who would be tented nearby should high water cause them to leave their homes hurriedly. Plans were made with many families whereby with a few hours' notice they would move to homes of neighbors above the danger level. Fortunately, emergencies did not actually arise which endangered life or property.

Methods employed.

Information was secured regarding the families remaining on TVA land until September 1935 and a grouping was made according to their removal and relocation needs. Tenant and owner families contacted by the Authority were each divided into five classifications as shown below:

Resources of families in Norris Reservoir area September 1935

	Owner	Tenant	Total
Farm families with adequate resources	388	179	567
Nonfarm families with adequate resources	31	105	136
Farm families needing assistance	78	140	218
Nonfarm families needing assistance	6	39	45
Families totally dependent upon assistance	6	44	50
Total	509	507	1,016

Upon making contacts, workers counseled with the families, discussed their problems, and helped them in every possible way to make use of all known resources. Many were referred to local cooperating agencies for aid; transportation to find new homes was offered to some; while to others trucking facilities, tents, and salvaged building materials were offered through the family removal organization. Threats of legal eviction were made to a very few; 5 families of a total of 2,899 were forced through court action to vacate. The material assistance given may be summarized as follows:

	Number of families
Aid from Resettlement Administration	39
Temporary loan of tents	45
Receiving salvaged building material	72
Trucking service for moving	156

Close cooperation existed continually between the Authority's reservoir family removal organization and the University of Tennessee's relocation service. In addition to relatives, neighbors, and friends of the reservoir families, other agencies cooperated as shown in figure 238.

Results accomplished.

The first concern was the removal of families from the zones to be flooded, and by April 8, 1936, all families had moved from below

elevation 1,020. As the area above this elevation was not in immediate flood danger, family removal from it was gradual. By the spring of 1938, all of the 2,899 families had moved to new homes. The following shows the approximate resettlement locations:

	Percent
Within the same five counties	62
Adjoining Tennessee counties	20
Other Tennessee counties	11
Adjoining Kentucky counties	4
Other localities	3

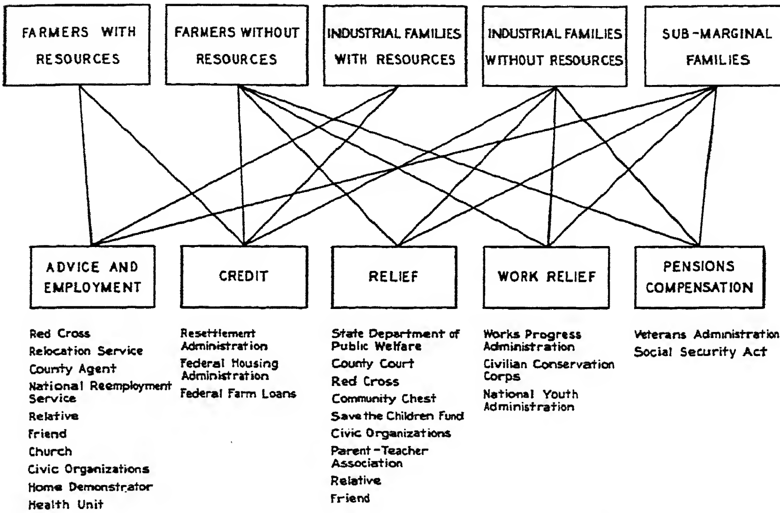

FIGURE 238.—*Agencies assisting in family removal.*

Cost of removal service.

The total TVA cost of the family removal service in all its phases was approximately $241,000—an average of about $83 per family moving from the reservoir. The service rendered the majority of the families consisted chiefly of guidance and cooperation rather than material assistance. Included in the total cost are payments made under a contract with the University of Tennessee Extension Division and payments under trucking contracts for moving 103 families.

RELOCATION OF CEMETERIES

One of the unique features of the construction of Norris Dam was the disposition of more than 5,000 graves affected by Norris Lake. This presented an extremely difficult and delicate problem due to the large number of graves involved and the religious and family sentiments of the inhabitants of the reservoir area.

Several hundred small family cemeteries on privately owned land and numerous church and community cemeteries were scattered

throughout the reservoir area. Most of the graves were unmarked, and many of them were very old—dating back before 1800. The people living in the valley maintained an unusual reverence for these graves, even the unknown and unmarked ones.

FIGURE 239.—*Representative of the better houses.*

FIGURE 240.—*Representative of the poorer houses.*

Although this was the largest known project of its kind ever undertaken, the work was finally conducted to the general satisfaction of all concerned and entirely free from litigation. Figure 241 shows representative views of the work.

Relocation policy.

It was agreed that the Authority should bear all reasonable expense of moving graves from the flooded areas below the 1,030 contour and from isolated areas to comparable burial places nearby,

FIGURE 241.—*Disinterment cemetery (above) and reinterment cemetery (below).*

that the removal should be made in accordance with the wishes of the nearest of kin, and that after relocation the Authority should have no further obligation for care or upkeep of the relocated grave.

In accordance with this policy, contracts were prepared covering a choice of three procedures which were offered to the nearest relatives:

1. The grave would be removed by a private undertaker of the relatives' choosing who would be paid by the Authority at the rate of $20 per grave.
2. The Authority would move the remains to the site selected by the relative.
3. The grave would be allowed to remain as it was and be inundated by the Norris Reservoir.

The greatest majority chose to have TVA forces make the removal. Of the total contracts obtained, only 22 were for allowing the graves to remain and be inundated; 341 were for removal by private undertakers; and 4,885 were for removal by TVA forces.

There were over 7,600 graves in cemeteries located along the margins of the lake which might be isolated although not submerged. Removals from such cemeteries were to be made entirely by the Authority's forces. A notice was distributed throughout the area advising the residents that removals from these cemeteries would be made upon request. When such requests were received, it was first ascertained whether or not the graves would actually be inaccessible, and in all, 836 removals were made from these isolated cemeteries.

Members of the churches having cemeteries affected by the creation of the reservoir felt that all of the several hundred unmarked graves which could not be identified should be moved. In all, 790 unidentified graves were relocated, of which 304 were unknown as to individual name but were identified as to family.

Surveys and identification.

Surveys were made of all cemeteries in the area, detailed plane-table surveys being made of those below elevation 1,030.

Plats were made of each disinterment and reinterment cemetery which show each grave location and number. The grave number and cemetery number are shown on all file records for easy reference. Two complete sets of all cemetery plats together with an index map were bound and filed with the grave removal records in two permanent depositories which are described later in the chapter.

Only a few of the original graves were marked with inscribed headstones, which made identification work difficult and required extreme care. Detailed questioning of the residents, checking of every clue, and the assistance of church representatives were combined to fix the identity of the graves. It was usually necessary to take the nearest of kin to the cemetery to identify the graves.

Reinterment cemeteries.

Many churches decided to join together and establish memorial cemeteries. This was accomplished by turning over to a board of trustees the proceeds from the sale of their original cemetery lands. With such funds, several tracts of land were purchased for new cemeteries. Reinterments from cemeteries of churches contributing to these funds were allowed free of charge; other reinterments were allowed upon payment of 50 cents for each grave. These memorial cemeteries and the number of reinterments were:

Baker Forge Memorial	1,879
New Loyston Memorial	1,093
Big Barren Memorial	948
Indian Creek Memorial	308
New Indian Creek	161

These 5 cemeteries provided for the reinterment of 4,389, or 84 percent of the total. The other reinterments were made in 89 small private and church cemeteries.

Relocation method.

Relocation of graves by TVA forces was conducted in a simple yet dignified manner. Workers were enlisted from the local area and in all cases were required to show proper reverence in the performance of their work.

The remains of burials more than 10 to 12 years old usually consisted of little more than the large bones of the body, a little dust, some articles of jewelry, and fragments of clothes or shoes. Usually the caskets more than 10 to 15 years underground were practically decomposed except for the nails or other metal parts. If possible the caskets were removed from the grave intact and placed in new boxes. When this was impossible, the remains, including all bones, dust, articles of jewelry or clothing, and any remaining parts of the original casket, were carefully placed in the new casket in the same position as found in the grave. An inventory was made of the remains and all articles found in each grave. Inexpensive wooden boxes were used as reinterment caskets. A few metal caskets were found intact, most of which had been furnished by the Government for burial of World War veterans.

The head end of the casket was marked, and all reinterments were made with the head toward the west in accordance with the custom of the people of the area. A simple metal grave marker with card insert was placed at the foot of each grave for temporary indentification.

All markers or monuments from the original graves were moved. These included inscribed headstones and footstones, native hand-cut stones, stone fragments and natural rock without inscriptions, wooden markers, and fences. Skilled stone workers were employed to handle the relocation of all large monuments.

Reinterment services were held when requested, but this was seldom. The nearest of kin were offered transportation to the cemeteries to witness removal and reinterment by TVA forces. Frequently the relatives requested other persons to witness the reinterment for them.

There were no cases where special care or protection was required to prevent contracting diseases, and no ill effects were suffered by the men performing the work.

Grave and cemetery records.

Detailed records kept in duplicate of all work in connection with each grave and cemetery included the following information:

1. Identification information and history of grave, including photographs of tombstones or markers that were indicative of identification.
2. Agreement for removal or to allow the grave to remain.
3. Reinterment data, including identification and inventory of remains.
4. Record of original and reinterment location, including cemetery plats.

One set of these records is filed in the TVA library in Knoxville as a permanent Government record, and a duplicate set is available to the public at the University of Tennessee library in Knoxville. A record of all removals was also furnished the State Health Department as required by the special regulations for the grave removals.

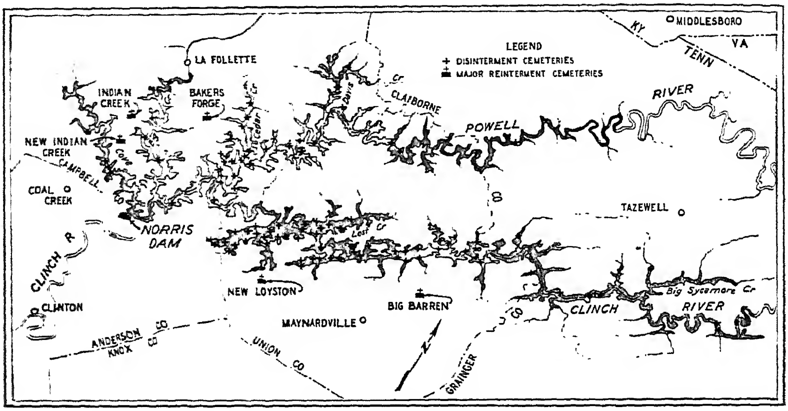

FIGURE 242.—*Disinterment and major reinterment cemeteries.*

TABLE 93.—*Summary of cemetery relocation data and costs as of Dec. 31, 1937*

Relocation data	Submerged area	Marginal area	Total
Cemeteries located	107	203	310
Cemeteries surveyed	107	125	232
Cemeteries in which graves were disinterred	107	31	138
Graves located	4,405	7,600	12,005
Graves relocated	4,383	843	5,226
Graves left to be submerged	22		22
Relocations by undertakers	341		341
Relocations by TVA forces	4,042	843	4,885
Total graves relocated	4,383	843	5,226
Reinterment cemeteries:			
Memorial (4,389 grave reinterments)			5
Others (837 grave reinterments)			89

Cost of relocation work	Total cost exclusive of general expense	Cost per grave
Preliminary work: Investigations, surveys and maps	$35,535.38	[1] $6.80
Removal work:		
By TVA forces (4,885 graves)	73,613.98	15.05
By undertakers (341 graves)	6,820.00	20.00
Total removal work	80,433.98	15.39
Total relocation cost	115,969.36	22.19

[1] Cost of surveying and mapping 12,005 graves divided by 5,226 graves actually removed.

Cost of cemetery relocation work.

To complete the entire program it was necessary to survey approximately 12,000 graves, 4,400 in submerged and 7,600 in marginal area, at a cost of $35,535.38. Table 93 shows the total cost of removals to be $80,433.98, making the average cost per grave $15.39. It is interesting to note that the grave removals by TVA forces cost less than those by private undertakers.

REMOVAL OF FORESTS, FENCES, AND STRUCTURES

Clearing the reservoir of forests and structures presented the usual and expected clearing problems. The mountainous area to be flooded was heavily and thickly timbered and the valleys were narrow with steeply sloping sides. Some 2,000 independently owned areas were cleared. The clearing of hedgerows and clusters of trees in pastures and around homes about equaled the timber cleared which stood in solid blocks of woodland. Hardwood timber of many species prevailed, sometimes in the virgin state.

It was decided that all of the brush and timber between elevations 1,020 and 940 be completely cleared by cutting and removing, or burning. In the area below elevation 940 the timber that projected above elevation 940 was to be felled and wired in place. The work was completed well in advance of rising water in the 26 months between February 1934 and April 1936. No fatal accidents occurred in connection with clearing operations.

Reservoir clearance organization.

The clearing work was directed from an office in Knoxville, Tenn. A superintendent, working from a field office in the area, was in direct charge of the clearing activities. A chief field clerk, reporting to the superintendent, was in direct charge of clerical work, such as timekeeping, pay rolls, bookkeeping, and handling of warehouse supplies. Time checkers, in addition to checking time, acted as contact agents between the various units and a centrally located warehouse.

The superintendent had 2 assistants to supervise clearing and 1 assistant in charge of timber conservation and burning. Standard clearing units consisted of 60 men under the supervision of a unit foreman. The units were divided into 3 groups: first, a bush and ax group cleared the area of brush and small timber; next came a saw group which felled the heavy timber; and finally, the last group piled the timber for burning. The work of each of the groups was supervised by a subforeman. A saw filer assigned to each unit kept the edged tools in good condition. The saw filer also served as clerk in charge of a small "field box library"[6] containing a maximum of about 50 volumes. These books, containing the best type of reading matter, were changed at frequent intervals. Each clearance unit formed a complete organization and worked independently.

Mobile first-aid stations were maintained and a safety instructor, fully familiar with the hazards of the work, was selected from among the unit foremen to conduct safety work.

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[6] Munn, R. Russell, Saw Filers and Book Boxes, Library Journal, 60: 720, Sept. 15, 1935.

Method of operation.

At the peak of operations 1,100 men were employed, practically all of whom were recruited from the immediate region. The work was divided into 4 major operations: regular clearing, clearing along stream banks, clearing below the draw-down, and burning of debris. A few crop damage claims occurred because the former owners were permitted, under the terms of purchase and sales contract, to occupy and cultivate their lands until December 1935. These were held to a minimum and the aggregate amounting to only $810.65 was surprisingly small.

Regular clearing.—In the draw-down area, the area between elevations 940 and 1,020, clearing consisted of cutting, piling and burning or removing all timber and debris.

FIGURE 243.—*Field box library.*

Clearing along stream banks.—This work was done with the aid of a 4-ton truck, equipped with a winch and niggerheads. Trees that had fallen into streams were thus drawn out for conditioning.

Clearing below the draw-down area.—Timber which projected above elevation 940 was cut and wired in place. No. 8 black annealed iron wire and ⅜-inch and ½-inch stranded black iron wire were used to anchor the timber to stumps.

Burning of debris.—All of this work was done by force account. A definite hazard resulted from burning operations and for this reason they were kept under careful control. Before burning any area, the upper edge was carefully raked and patrols were stationed at strategic points with fire-fighting equipment. In emergencies, assistance was given by the surrounding units. In burning more than 18,000 acres, less than 50 acres of woodland above elevation 1,020 were burned.

Use of machinery.

Machinery was not used extensively in clearing this reservoir, partly because of the region and partly because of the limitations of machinery. The steeply sloping banks with only a narrow strip to be cleared along either bank did not lend themselves to the use of ordinary clearing equipment. Power saws were carefully investigated but were not used as it was felt that they had not been developed sufficiently.

A proposal was made to use skidders in some of the flatter regions. This machinery with the aid of tractors was to draw untrimmed, felled trees from a 2- or 3-acre tract into a centrally located large pile for burning. Tests of such equipment, made in the Wheeler

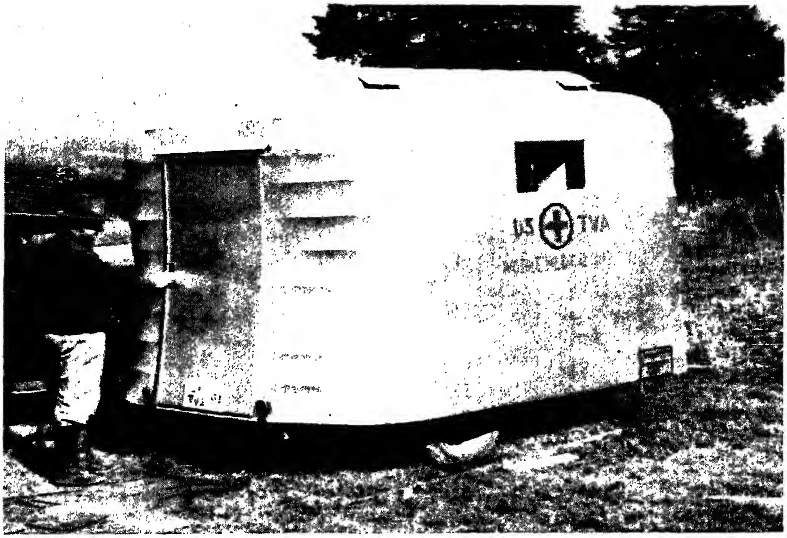

FIGURE 244.—*Mobile first-aid station.*

Reservoir by manufacturers' representatives, revealed that their clearing costs were about double the costs of unit crew methods used by the Authority. The scheme failed because of unpredictable difficulties and delays experienced in handling cables and getting the trees to "ride" up the pile without gouging into it.

A power unit was necessary for pulling large trees out of streams into which they had been felled. The most powerful and practical equipment found for this work was tractors equipped with booms and stiff-legs developed by the Authority for the Wheeler Reservoir. Unfortunately these crawler-type, slow-moving machines could not be used in the Norris Basin which required extremely mobile and fast-moving equipment. Therefore a 4-ton truck equipped with a drum and winch was used.

Obstacles encountered.

The steep mountain sides made clearance work both difficult and hazardous. At times men were suspended by ropes over cliffs to cut timber. More than 50 percent of the clearing had to be done on lands before titles were secured. To do this, permission had to be obtained from the owners who frequently requested that fences be left and that orchard and certain shade trees be left uncut. All such requests were complied with, necessitating a follow-up program after the lands were purchased. This follow-up work began in December 1935 after other clearance work was completed. Several crews of from 15 to 25 men each, operating from trucks, covered the entire area and removed the remaining trees, fences, and buildings.

Occasionally clearing forces were ordered off the lands at the point of guns. In most of these cases the superintendent was able to settle the matter amicably and arrange for clearance work to proceed.

Impoundage of the reservoir began on March 4, 1936, and a fleet of small boats was organized to dispose of the flotage that might occur. This flotage, however, amounted to less than 1 percent of the cleared material. The efforts of this organization were engaged entirely in removing quantities of drift that had been swept into the reservoir by run-off waters from above the 1,020-foot contour.

Merchantable timber.

Before an area was cleared, an estimate was made of all merchantable timber that might be made available. Such timber was cut into stock lengths by the clearance forces, later to be manufactured into lumber by contract mills selected on a competitive basis. This lumber was used for TVA construction purposes. Timber that could not be used by the Authority was offered for sale and sold in the log to the highest bidder. The following quantities of timber were utilized from the area:

Used by TVA		Sold to outside parties	
Lumber___ft. board meas___	4, 555, 458	Logs_____log-scale measure__	40, 506
Cedar posts_______number__	102, 605	Pulpwood___________cords__	77
Cedar poles__________do____	2, 981	Dogwood____________do____	20
Pine logs____________do____	3, 025	Oak and cedar logs__number_	283
Shingle boards ______do____	60, 500		

Fences and buildings.

In addition to completing the removal of trees left for various reasons, the clean-up crews completed the removal of all buildings and fences. Although each former owner was permitted to remove his buildings prior to December 1935, many structures in dilapidated condition were left. Approximately 4,000 old buildings and building foundations had to be cleaned up and burned. Structures left on condemned property were advertised for sale and removed by purchasers. However, the foundations of these buildings had to be cleaned up.

Production and costs.

The man-days per acre required for clearing the wooded areas were as follows:

DRAW-DOWN AREA (13,732 acres)		BELOW DRAW-DOWN AREA (4,961 acres)	
Cutting and piling	16.2	Cutting	2.8
Burning	2.5	Burning	0.2
		Wiring	6.3
Total man-days per acre	18.7	Total man-days per acre	9.3

The total direct cost for reservoir clearance amounted to $1,140,358.32—$1,094,027.86 being for labor and $46,330.46 for materials and depreciation. Special studies revealed the clearing cost per acre to be approximately as follows:

Cutting, piling, and burning in the draw-down area

Labor	$64.23
Materials and depreciation	1.62
Reservoir clearance overhead	6.49
Total cost per acre	72.34

Cutting, burning, and wiring below the draw-down area

	Cutting	Burning	Wiring	Total
Labor	$12.12	$0.66	$21.26	$34.04
Materials	.53		4.79	5.32
Reservoir clearance overhead	1.25	.06	2.57	3.88
Total cost per acre	13.90	.72	28.62	43.24

ARCHAEOLOGY

The archaeological surveys and excavations conducted in the Norris Reservoir area were made in an attempt to bring to light before the flooding of the reservoir detailed evidence of prehistoric occupation which might be valuable in solving some of the many archaeological problems of the southeastern United States. Such an investigation was deemed necessary because it was felt that the early Indians had used these rivers for highways and might have left important traces of their life in the section.

The first step in the preservation of the prehistoric material was taken early in December 1933 at a conference attended by representatives of the Authority, the University of Tennessee, the University of Alabama, and the United States National Museum. Tentative plans were made and work was begun in January 1934, continuing until July of the same year. This work was under the direction of the head of the Department of Anthropology and Archaeology at the University of Kentucky. The survey revealed 23 sites showing definite evidence of prehistoric occupation. With the use of Civil Works Administration labor and State-financed relief workers, 20 earth mounds, 9 stone mounds, 4 village sites, and 7 caves at the 23 sites were investigated. Of these 29 mounds, 12 were burial mounds and 17 were associated with prehistoric structures. Evidences of 54 structures were found, 20 of which were thought to have been dwellings and 34 designated as "town houses." Seven of the town houses had suffered incomplete combustion and collapsed after being reduced to charcoal.

All of the artifacts recovered were deposited at the University of Tennessee for study and photographing. The skeletal material was shipped to the Department of Anthropology and Archaeology at the University of Kentucky for restoration, study, and report; and samples of potsherds from all sites were sent to the ceramic repository of the University of Michigan for study and report. The results of these studies and a detailed description of each site form a comprehensive report that has been issued as Bulletin 118 [7] by the Bureau of American Ethnology.

The work in the Norris area indicates that at least two new culture complexes were discovered and their material culture traits were established. Of particular significance in working out these traits, known as those of the "large-log" town house and the "small-log" town house peoples, were the remains of prehistoric buildings identifiable by imprints or molds of the original posts as left in the various

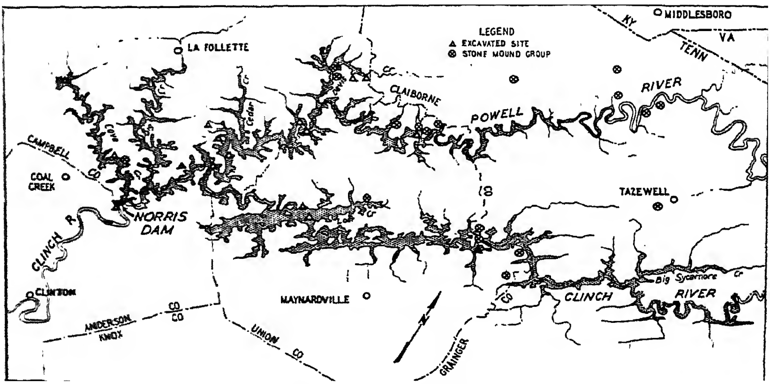

FIGURE 245.—*Archaeological survey.*

layers of the mounds. In some instances even parts of the logs themselves were recovered. It is possible that with the aid of dendrochronological comparisons, made with the annual rings of the oldest living trees of the same species in this region and from logs taken from old buildings in the neighborhood, the chronological periods in which these people lived may be identified.

It has been possible to identify three classes of civilization in the area, one low in cultural development and two well up in barbarism. For the latter two it was possible to establish cultural traits. These are grouped in the following sections:

Small-log town house people.—These people built extensive villages having rectangular dwellings of small logs covered with cane and grass thatch over which they often placed a coat of earth. On the floors of these houses were found raised square clay platforms from 2 to 6 inches above floor level. Four circular fire basins were usually

[7] Webb, William S., An Archaeological Survey of the Norris Basin in Eastern Tennessee, bull. 118, Bureau of American Ethnology, 1938.

found on these square platforms indicating ceremonial significance. A clay seat of two steps erected at one end of the building constituted a place for the presiding officer. Woven and plaited matting upon which the participants sat in council covered the floors of these structures. There was evidence that these people practiced agriculture, cooked food, wove textiles, and made shell-tempered pottery. The method of disposal of the dead was not found as no cemeteries or burial places could be located.

Large-log town house people.—These people also built extensive villages with rectangular buildings and rectangular town houses. Logs as large as 14 inches in diameter were used in their construction. Apparently these people did not burn their houses ceremonially as the small-log town house people seem to have done. Neither did they use square altars or fireplaces or construct clay seats. Instead they used a circular fire basin. They made a distinctive type of pottery and often buried their dead in pits dug in the floor of the town house. In such pits the body was placed in a sitting posture, usually accompanied by a great store of artifacts. The grave pit was covered with clay and the clay floor of the town house restored and its use continued. Evidently only notables received such burials in town houses. The majority of the people doubtless were buried in cemeteries; however, all traces of such cemeteries in the Norris Basin have been destroyed by intensive cultivation.

HIGHWAYS AND RAILROADS

The creation of the Norris Reservoir necessitated adjustments of utilities, highways and railroads, and telephone and telegraph lines, amounting to $3,156,179.57 for total direct costs, the principal adjustments being for highways and railroads. Much negotiation was necessary, resulting in some cases in compromise figures for adjustments, but in no cases were litigations necessary. For each adjustment a contract was executed by the Authority and the agency or utility affected covering the adjustment to be made.

HIGHWAYS

Highway adjustment contracts were made with the State of Tennessee and the counties of Campbell, Claiborne, Grainger, and Union. Adjustments in most cases consisted of relocation of the highways affected which in some cases increased the length of travel between communities. In most instances, however, a road of better standards was built to compensate for this additional distance. In each of the contracts it was stated that for the considerations made the Authority would have no further liability or maintenance claims for highways being inundated.

It was found more economical to buy certain isolated farms above the taking line than to construct access roads to them. This was particularly true in the central peninsular region between the Clinch and Powell Rivers.

Table 94 shows the state and county highways affected and relocated in the reservoir area.

TABLE 94.—*State and county highways affected in the reservoir area*

	Miles	Total cost
State highways:		
Highways affected	22.8	
Reconstructed by TVA	12.0	
Reconstructed by State	4.5	
Total direct cost		$1,018,318.09
County highways:		
Highways affected	278	
Reconstructed:		
Principal county roads	64.3	877,878.88
Tertiary roads	23.8	85,296.24
Adjustment for release of flood liability		141,000.00
Bridge removals		26,583.09
Total direct costs		2,149,076.30

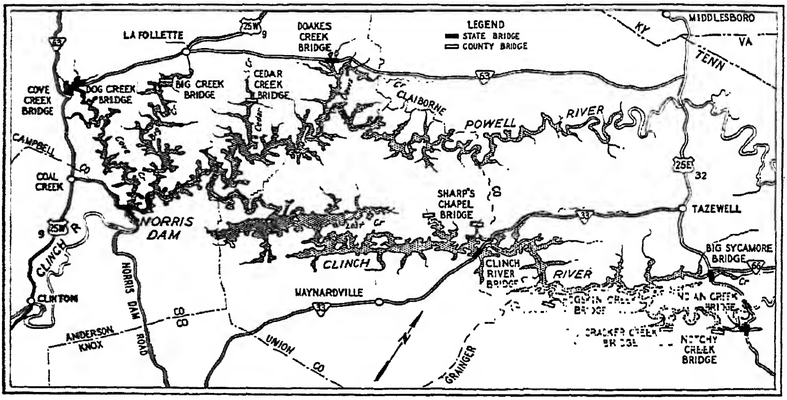

FIGURE 246.—*Principal highway bridges.*

State highways.

Some of the State highways to be relocated were of inadequate width and had obsolete alignment. Although the Authority's obligation extended only to the replacement of similar roads, it was agreed by the Authority and the State that it would be undesirable to reconstruct on the same standard. The contract with the State, therefore, provided that on certain roads the Authority would construct roads of greater width and higher standards of alignment, and that in compensation for the extra expense, the State would reconstruct certain sections of flooded road at its expense, and would release the Authority from responsibility for all of the highways that were flooded. A part of the Authority's work and expense was to secure all necessary rights-of-way for the construction of state highways to be relocated by either the Authority or the State. Minimum roadway elevation was 1,030. Minimum paving width was 18 feet on highways and 24 feet on main bridges. Rights-of-way were a minimum of 80 feet in width for main roads and 60 feet for secondary ones. Of the 16.5 miles reconstructed, 1.3 were surfaced with concrete, 10.6

with bituminous top on a 6-inch waterbound macadam base, and 4.6 with traffic-bound gravel.

The most important bridge constructed is the Clinch River Bridge on Tennessee State Highway 33. It had a total length of 1,915 feet and the direct construction costs amounted to $349,795.04. The principal bridges that were relocated in the area, as shown in figure 246, for state highways were:

Location	Type bridge	Number spans	Length feet
Built by the Authority:			
Highway 33 at Clinch River	Steel trusses with concrete approaches	4	1,915
Highway 9 at Cove Creek	Concrete deck girder	3	162
Highway 9 at Dog Creek	do	2	108
Built by the State of Tennessee:			
Highway 63 at Doakes Creek	do	8	242
Highway 66 near Sneedville	Concrete box	2	23
Highway 32 at Big Sycamore Creek	Concrete deck girder	8	212
Highway 32 at Indian Creek	do	5	132

County highways.

As explained previously, many of the inundated county highways were abandoned. Studies were made to balance properly the cost of relocation against lands that might be purchased, and in many cases it was found to be more economical to purchase additional land than to build a road to it. This was especially true with tertiary roads serving only one or two farms. All agreements with counties were based on the general principle of replacing the existing facilities as nearly as feasible either by raising or relocating. Although in some instances the new roads necessitate longer travel, they provide the residents with improved highways which compensate for the additional distance.

The Authority made surveys, prepared plans, secured necessary rights-of-way, supervised construction, and removed existing bridges where necessary. The completed highways were turned over to the counties with no further responsibility for upkeep or maintenance by the Authority.

In addition to the relocation of highways in Union County, a settlement of $141,000 was made because of the number of highways and bridges that had to be abandoned. This settlement permitted the county to retire the outsanding highway and bridge bonds on facilities that were flooded.

All county highways were constructed to a minimum elevation of 1,028, with roadbed widths varying from 10 feet for minor tertiary roads to 20 feet for principal county roads. The surfacing was bituminous treated or traffic-bound macadam or gravel. A few minor tertiary roads were not surfaced and varied in width from 10 feet for the narrowest tertiary roads to 18 feet for principal county roads. They were constructed to a minimum elevation of 1,030. Forty-three reinforced concrete culverts were constructed in sizes varying from 4 by 3 feet to 22 by 18 feet. The alignment and gradient of the tertiary roads followed in general the existing ground.

Where possible, abandoned bridges were re-erected at relocation sites where new bridges were needed.

TABLE 95.—*County bridge construction*

Road location	County	Number of spans	Length	Roadway width
			Feet	*Feet*
La Follette-Ageo	Campbell	4	296	18
Primroy-Cedar Creek	do	5	355	14
Notchy Creek	Grainger	1	60	12
Cracker Creek	do	1	63	18
Hogskin Creek	do	1	160	16
Sharps Chapel	Union	1	63	18
Do	do	1	45	18

Fifteen timber bridges were also built, most of which were on tertiary roads.

TABLE 96.—*County highway construction*

County	Principal county roads	Tertiary roads	Total
	Miles	*Miles*	*Miles*
Campbell	36.3	8.2	44.5
Claiborne	7.6	8.0	15.6
Grainger	12.3	.4	12.7
Union	8.1	7.2	15.3
Total	64.3	23.8	88.1

Bridge removal.

Those bridges which when flooded would have affected navigation during normal operation pool elevation were removed by the Authority. As stated previously, where possible, they were moved to other

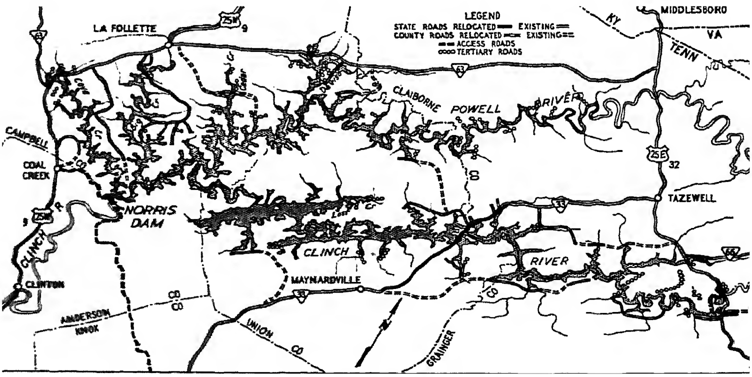

FIGURE 247.—*Location of highway relocation work.*

locations to be reused. Those not used were given to the county from which they were removed. A total of 33 bridges was thus removed at a cost, exclusive of indirect construction expense, of $26,583.09.

The demolition of three concrete bridges was carried out by the Corps of Engineers, the United States Army Engineer Board having

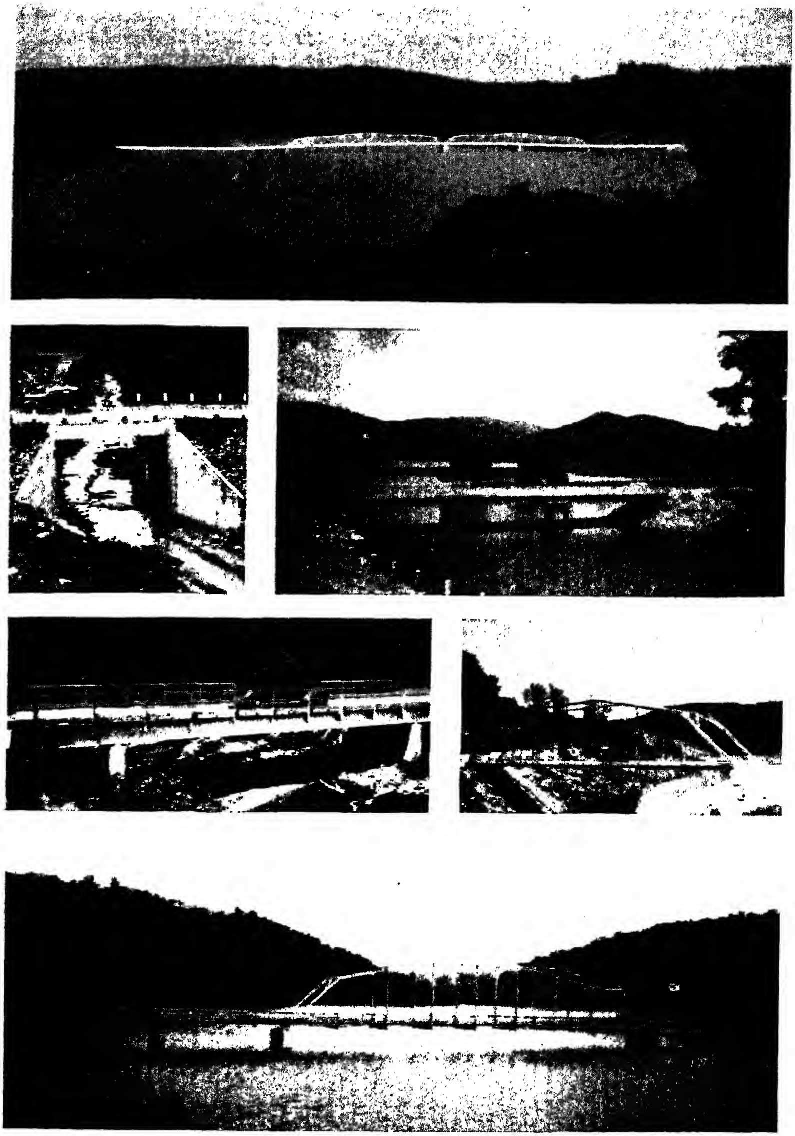

FIGURE 248.—*Typical new state and county highway bridges. A. Clinch River Bridge, State Highway 33. B. Little Barren Creek Bridge, Peninsula Road. C. Cove Creek Bridge, State Highway 9. D. Notchy Creek Bridge, Henry's store to State Highway 32. E. Hogskin Creek Bridge, Black Fox Creek Road. F. Cedar Creek Bridge, Primroy-Cedar Creek Road.*

requested the use of the structures for making comparative tests on the effect of various types of explosives used in military demolition. The largest of these was the Walker's Ford Bridge built in 1921 on the Hickory Valley road between Maynardsville and Lone Mountain and consisted of eight 60-foot concrete arches carrying a 16-foot roadway. The Sill Bridge of similar construction crossed the Clinch River on the road between Loyston and Sharp's Chapel, Tenn. It consisted of four 88-foot arches and had a roadway of 14 feet. The smallest of the three was the Loyston Bridge, located in Union County, between Loyston and La Follette, consisting of two 156-foot open arches and had a roadway width of approximately 15 feet.[8]

RAILROADS

The creation of the reservoir necessitated adjustments on two branch lines of the Southern Railway and on one line of the Louisville & Nashville Railroad. The complications that arose required much planning and negotiation with railroad officials and resulted in individual compromise agreements in each case. Among other factors, studies were made of the traffic on each branch line of the Southern Railway to determine whether the line should be abandoned or to what extent relocations were economically justified.

The railroad officials at first maintained that the Authority should bear all expense and that the work should be entirely in accord with their plans and specifications. From a legal standpoint, however, it appeared that the railroad should bear the expense of crossing navigable streams. The Clinch River was considered navigable, and since raising a bridge across it was a considerable part of the work, the question of responsibility for relocation expense became quite involved. The negotiations were further complicated by the fact that lines of these two railroads were also affected by the concurrent construction of the Wheeler Reservoir, and the railroad officials insisted upon completing arrangements satisfactory to them in both reservoirs before entering into contracts for any adjustments. Much negotiation was necessary to determine the elevation to which relocation should be made; and it developed that each new line, principally because of usage, was built to a different minimum elevation. It was agreed that relocated lines should be the equivalent of the existing ones as to curvature, grade, and roadbed, although the new roads were actually somewhat superior to the previous ones. It was also agreed that trains must not be delayed while the work was progressing.

The total direct costs of all railroad relocation and adjustment are given below:

Southern Railway relocation	$884, 361. 15
Louisville & Nashville Railroad relocation	108, 598. 26
Total direct costs	992, 959. 41

Middlesboro branch of the Southern Railway.

This line, between Knoxville, Tenn., and Middlesboro, Ky., is single track of second-grade railroad construction. Although traffic was not heavy, the line served a wide area. Relocation of it, therefore,

[8] Engineering News-Record, 116 : 806–807, June 4, 1936.

was economically desirable. From the first conference on December 11, 1934, until the contracts were signed on June 28, 1935, constant negotiations were underway.

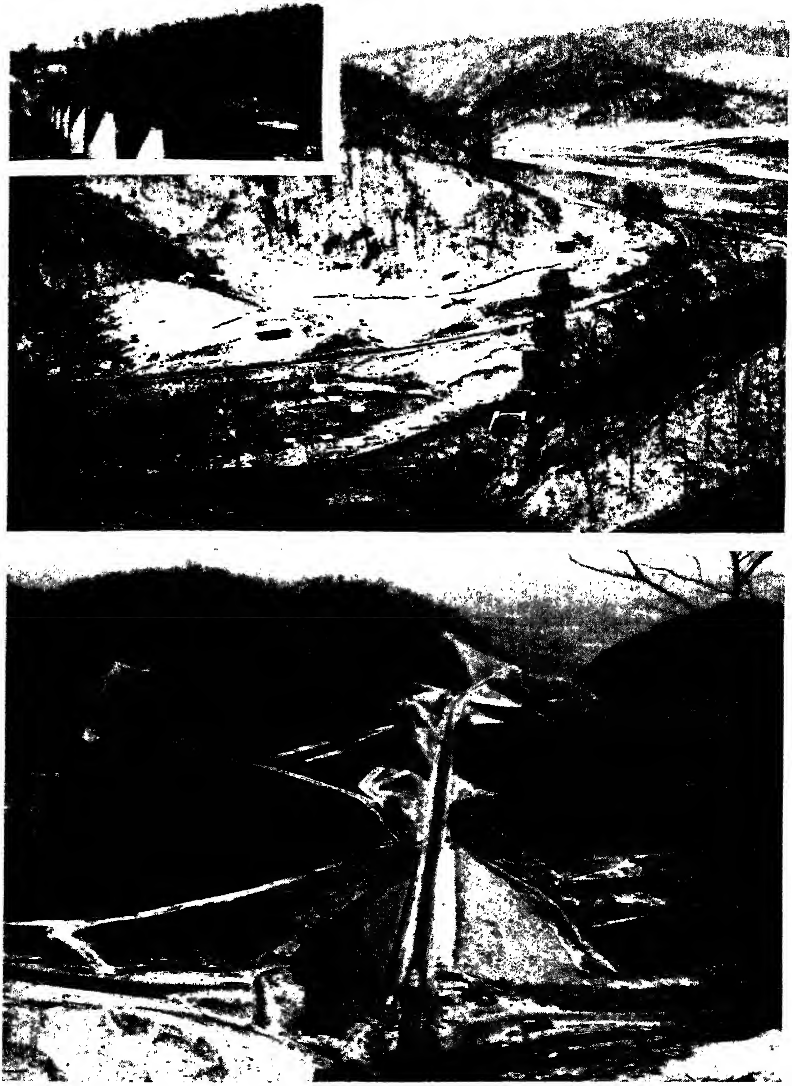

Figure 249.—*Railroad relocation. A. Clinch River Bridge. B. Sycamore Creek Bridge piers under construction. C. Approach to Sycamore Creek Br*

The Authority paid all expense of relocation and the railroad accepted elevation 1,038 for the lowest point in the roadbed and 1,037 for the lowest point of steel on bridges as adequate protection. It was considered that a clearance of 16 to 20 feet above the crest of the spillway would be adequate for navigation. The actual clearance is about 17 feet.

Plans.—The selected location eliminated a considerable loop of the existing track by means of two tunnels totaling 910 feet in length, one south of the Clinch River and one north of Big Sycamore Creek. Two bridges with a total length of 1,429 feet were required to replace existing structures which were too low. One was across the Clinch River and the other was across Big Sycamore Creek. About 3 miles of new track were required and the line was shortened by 2,758 feet. The old and new locations are shown in figure 250. The three 150-foot deck trusses of the old Clinch River bridge could not be reused, so new structures were designed of 100-foot deck girders on solid masonry piers. The same type of structure was also used for the Big Sycamore Creek crossing. This design, although providing more durable bridges and less curvature and grades, was the most economical that could be devised. The Southern Railway requested the Authority to prepare all plans and specifications and to supervise the construction work. Accordingly, detailed plans were made and became a part of the contract before construction work began or before invitations to bid were published. As the time before flooding was short, the Authority built by force account the foundations for the two bridges as well as all other drainage structures. Work was planned so that immediately after the contract was signed work actually began on the pier foundations and invitations to bid were distributed. The contract for grading and tunneling was awarded to Nello L. Teer of Durham, N. C., and the fabrication and erection of the two bridges was awarded to the McClintic-Marshall Corporation, a subsidiary of the Bethlehem Steel Corporation.

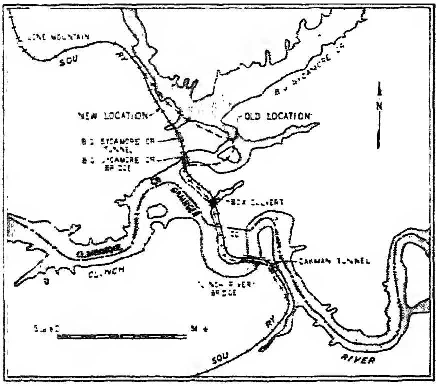

FIGURE 250.—*Old and new location of Middlesboro branch of Southern Railway.*

Construction.—The severe and unusual winter and unanticipated slides delayed construction considerably. For a while it appeared that this construction work would delay the filling of the reservoir. However, traffic was diverted on April 10, 1936, 12 hours before the old track was flooded. A supplementary contract was let to McClintic-Marshall Corporation to remove the steel superstructure of the old Clinch River bridge so as not to interfere with navigation. Time

did not permit complete salvage of the spans and they were left on the bed of the river. The plate girders of the Sycamore Creek bridge were inundated but later salvaged when the lake was drawn down sufficiently.

Costs.—The total cost of the relocation work was as follows:

Work done by TVA forces	$329, 133. 53
Nello L. Teer	413, 258. 88
McClintic-Marshall Corporation	111, 288. 26
Southern Railway Co.	30, 680. 48
Total direct construction cost	$884, 361. 15

Maintenance.—The contract between the Authority and the Southern Railway provided for maintenance work on the fills by TVA for 2 years after the water reached elevation 1,020. As the water first reached this elevation on January 23, 1937, the period of maintenance extended to January 23, 1939, and an estimate of $30,000 per year was set aside for this purpose. During the first year, after the water was around elevation 1,020 for about 9 months, only $593.34 had been spent for maintenance. During the spring of 1937, unexpected conditions, not likely to occur more than once in 25 to 50 years, brought Norris Reservoir to 1,031.07. After this flood had receded, an inspection of fills revealed no damage. Therefore, further maintenance should be negligible. At some future date, should the reservoir rise above 1,034 due to closure of the gates, the Authority is liable by contract for reimbursing the railroad for any damage that may occur.

Louisville & Nashville Railroad.

The main line of the Louisville & Nashville Railroad between Knoxville, Tenn., and Cincinnati, Ohio, is single track of first-grade construction, and carries heavy passenger and freight traffic. It crosses Cove Creek, an arm of Norris Reservoir, as shown in figure 251 on a 595-foot steel viaduct. The construction problems here were relatively simple as compared with the Middlesboro branch line of the Southern Railway. Work consisted only of supplying new foundations and partially rebuilding the existing steel viaduct.

Plans and negotiations.—As the existing concrete pedestals of the trestle would be entirely inundated and not open to further repair, it was deemed necessary to rebuild them entirely and carry the concrete above the pool elevation of 1,020. A factor that helped reduce the final adjustment but which complicated the negotiations was the simultaneous negotiations being made with the Southern Railway for abandoning their Vasper-La Follette branch. This is described later.

The contract stipulated that the Authority was to bear all the cost of the construction work to be done. The railroad prepared the plans and specifications and supervised the construction work.

Construction.—The contract for work was awarded to W. W. Boxley Co. The maximum continuous time allowed for replacement of steel without interrupting traffic was less than 3 hours. The new piers were constructed between the existing pedestals and were carried to a point where they would support three 100-foot steel girder

spans carrying the track. Girder spans were constructed on falsework parallel and adjacent to the existing spans. The supports for the old spans were cut with acetylene torches and removed with locomotive cranes. The new assembled spans were rolled into position and raised to proper alignment. Four spans were placed in this manner with traffic at no time being held up more than a few minutes.

Costs.—The total direct cost of this relocation work was $108,598.26. The Louisville & Nashville Railroad took advantage of the construction contract to fill out one end of the reconstructed steel viaduct at its own expense, but the other end was left on existing steel structures. No maintenance work was involved as the work was well above high water and not in danger of flood damage.

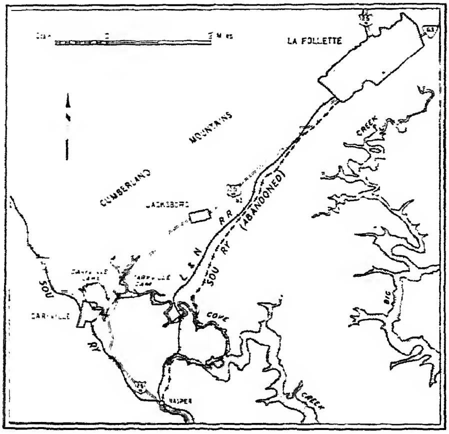

Figure 251.—*Location of Cove Creek brid; of Louisville & Nashville Railroad and abandoned Vasper-La Follette branch of Southern Railway.*

Vasper-La Follette branch of the Southern Railway.

This branch of the Southern Railway was a short single-track line running between Vasper and La Follette. It might be classed as second- or third-grade railroad construction with sharp curves and fairly steep grades. It was built chiefly for carrying coal, and, as this business had diminished in later years, maintenance work had been largely deferred. Traffic was so small that there was grave doubt as to the justification of the existence of the road. Two consulting engineers, specialists on railroad economics, confirmed the Authority's conclusions that the branch line was losing money and it would be to the best interests of the Southern Railway to abandon it.

In the meantime, the Authority, in negotiation with the Louisville & Nashville Railroad for an adjustment at Cove Creek, worked out an arrangement whereby that railroad agreed to carry the traffic of the Vasper-La Follette branch at a price quite advantageous to the Southern Railway. The proximity of the two lines is shown in figure 251. Automatic signals and new connections between the lines at Coal Creek and La Follette would have been necessary. The Authority was to stand the expense of this work, estimated at $50,000. The Southern Railway, however, finally objected to using a competitor's track, and these negotiations never materialized.

After considerable negotiation a settlement was reached whereby the Authority paid a lump sum of $125,000 in lieu of all damages. The railway deeded to the Authority a portion of its right-of-way on this line, the most valuable part of which was the station and siding at La Follette. The road was used until high water covered

the tracks late in 1936. Abandonment of this line was formally granted by the Interstate Commerce Commission on January 22, 1937.

There is still some dissatisfaction on the part of a few shippers who suffered because of abandoning this line. These may in time be adjusted, however, as the Authority stands ready to deed to the Louisville & Nashville Railroad the right-of-way acquired from the Southern Railway so that the Louisville & Nashville Railroad can establish service to these shippers.

RELOCATION OF ELECTRIC POWER, TELEPHONE, AND TELEGRAPH LINES

Liability for damages in connection with telephone, telegraph, and power lines arising from the construction of Norris Dam, although not amounting to large sums, was very involved and required considerable negotiation. In no case, however, were condemnation proceedings or other litigations required. Amounts decided upon were

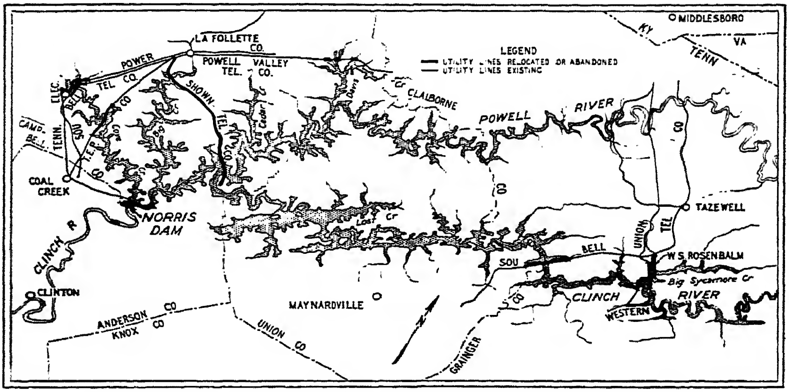

FIGURE 252.—*Communication and power lines that were relocated.*

in most cases compromise figures adjusted for conformity with reasonable estimates and the Authority's liability, but all agreements were reached without any semblance of coercion. The contract in each case provided that the company would protect its property from floodwaters of Norris Reservoir and waive all claims for damage arising from such flooding in consideration of payment received.

Clearances over the reservoir for telephone, telegraph, and low-voltage distribution lines were set at 30 feet above normal pool elevation; and clearances for high-voltage transmission lines were set at 67 feet above normal pool elevation. In most cases the companies preferred to make the necessary adjustments with their own forces rather than have the Authority perform the work or let it by contract.

The total adjustments of the six companies amounted to $18,250. The adjustment required for each line is discussed separately and figure 252 shows the geographical location of the lines affected.

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SOUTHERN BELL TELEPHONE & TELEGRAPH CO.

This company had lines crossing the reservoir in two locations that were flooded:

1. The Corbin-Knoxville line crossing the Clinch River near Walkers' Ford, which involved the relocation of about 2 miles along Flint Creek and Bear Creek.

2. The Danville-Knoxville line at Caryville which involved the relocation of about a mile and a half of line along Cove Creek.

The Authority paid the company $7,241 and in consideration was released from all claims for damages. The cost of the submarine cable crossing the Clinch River was not considered a liability of the Authority, and this cost of approximately $1,000 was borne by the telephone company. The work was completed and approved early in 1936.

WESTERN UNION TELEGRAPH CO.

A telegraph line, parallel to the Middlesboro branch of the Southern Railway, was located below the pool level for about 2 miles near the Clinch River crossing. Since the line was on the railway right-of-way, the telegraph line adjustment depended largely upon plans for the relocation of the railroad. Negotiations and plans for the adjustment of the telegraph line were therefore developed simultaneously with the plans and agreement with the Southern Railway Co.

Original plans were to make a temporary relocation of a portion of the line to permit construction of the railway to proceed and later to make permanent relocation of the line. Estimates for these two phases of the work were made and compared favorably with the preliminary estimates. However, by combining the two phases a considerable saving was made, and the final cost was $6,192.16. The entire work was completed prior to flooding of the reservoir.

TENNESSEE ELECTRIC POWER CO.

The Tennessee Electric Power Co. transmission line between Coal Creek and Westbourne, which crossed Cove Creek near Lovely's Mill, was affected by the Norris Reservoir. This line was a 66,000-volt, 3-phase line with 2-pole *H*-frame structures. It was necessary to relocate about 1 mile of line including 28 structures.

Plans and estimates of cost of the relocation were agreed upon and arrangements made for the power company to do the work on a force account basis, submitting detailed bills to the Authority for payment in accordance with procedure followed on other work previously done for the Authority. The line was relocated on land owned by the Authority, for which a right-of-way was granted the power company, who in turn quit-claimed all rights they had in Authority land where the line was abandoned. The final cost of the work amounted to $3,682.51 and was found to be satisfactory after checking.

After notification by the Authority, the power company relocated a distribution line paralleling State Highways 9 and 63 at Caryville for a distance of several hundred feet where it was affected by the Caryville Lake. In the notification to the power company, the Authority disclaimed all liability for the cost of this relocation; however, after its relocation the power company presented the Authority with

a bill amounting to $1,159.74 to cover the cost of the work. The position taken by the Authority regarding its liability for this relocation has resulted in no settlement being made of this claim.

SHOWN TELEPHONE CO.

This privately owned telephone line extended from the Southern Bell Telephone Co.'s line near La Follette to a point near Agee on the Powell River, a distance of about 8 miles, and served eight subscribers. The reservoir flooded the land at two places for a total distance of about 2 miles; and in addition, several miles of the line were rendered useless because the connected parties were bought out by the Authority in acquiring reservoir land. The Authority paid the telephone company $640 to cover all damages and cost of relocation, including settlement for the abandoned section of the line; the company thereby released the Authority from all claims for damages arising from the construction of Norris Dam.

ROSENBALM AND RUNION TELEPHONE LINE

Near Lone Mountain, Tenn., there was a small privately owned telephone line about 2 miles in length which was not in use and was also in need of repair. During the relocation of the Southern Railway at the Clinch River crossing by the Authority, it was found desirable to use this telephone line in furnishing telephone service to the Authority railroad construction forces. Arrangements were then made for purchasing the line outright and repairing it for use by the construction forces. Thus two requirements of the Authority were settled at once.

POWELL VALLEY TELEPHONE LINE

This privately owned telephone line parallels Highway 63 from a point near La Follette to Wells Springs, a distance of about 9 miles. Sections of the telephone line and the highway were located below the pool level and their relocation was required before flooding of the reservoir.

The creation of the reservoir required relocation at three places for a total length of less than 1 mile. These adjustments were taken care of by the telephone line owner following lump sum payment by the Authority.

BACKWATER ADJUSTMENTS

Even though the Norris Reservoir had a shore line approximately equal to that of Lake Michigan, only two small towns—Loyston and Caryville—were affected by floodwaters. The former was completely inundated, and about one-third of the latter would be affected when the reservoir level was at the top of the spillway gates or higher. Being in a mountainous region, free from major industrial developments, the project was also spared backwater protection adjustments to industrial plants.

LOYSTON

Loyston, an unincorporated town of less than a hundred people, on State Highway No. 61 in Union County, was so located that protection against floodwaters was beyond the remotest possibility. The

entire town, consisting of a few country stores, churches, schools, and homes, was bought outright in the manner provided for other reservoir property acquisition. Most of the inhabitants in the town moved about 4 miles south on the same highway and formed a new community known as New Loyston.

CARYVILLE

Description of the town and the problem.

Caryville is an unincorporated town in Anderson County, 35 miles from Knoxville, just off United States Highway No. 25 which connects Knoxville and Cincinnati. It is situated on Cove Creek, now an arm of Norris Lake, and has a population of 1,100.

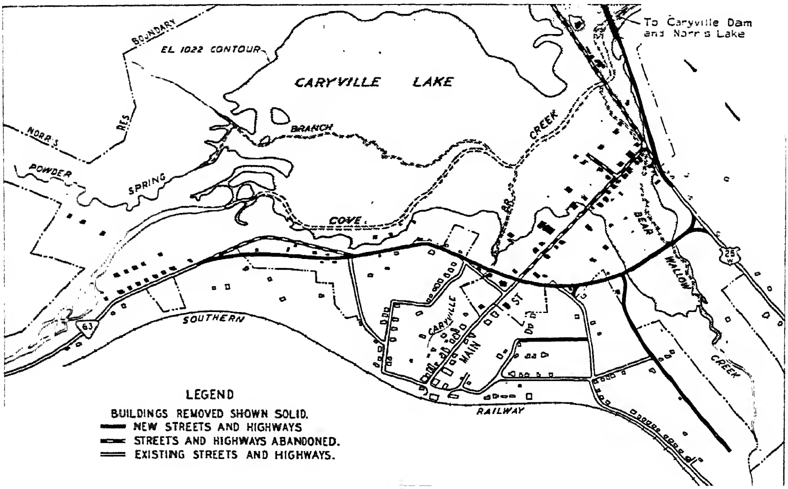

FIGURE 253.—*Caryville, Tenn.*

A water elevation of 1,011 or more would have flooded a portion of Caryville. As the top of Norris Dam gates was to be at elevation 1,034 with possible floods even higher, it was certain that Caryville would be troubled with reservoir water backing up Cove Creek. Substantial readjustment in the town and roads leading to it was necessary. Also, the fluctuation of the water level in the reservoir would expose unsightly mud flats which would depreciate property values.

Considerable study was given to the problem in a desire to provide better rather than worse conditions as a result of the Norris project.

The only industry in the immediate area is coal mining, and the coal deposits appear to be sufficient for about 100 years' supply at the present rate of consumption.

The street layout is shown in figure 253. About 3,000 feet of street were flooded with the reservoir at elevation 1,020, about 8,500

feet with the reservoir at 1,035, and about 10,400 feet at elevation 1,044. The estimated value of real estate and land values within the town below elevation 1,025 was $56,400; below elevation 1,035, $135,300; and below elevation 1,041, $174,500.

In making the studies, it was necessary to consider possible local floods from Cove Creek as well as backwater from Norris Reservoir. This creek drains 35 square miles, and floods from it have caused substantial damage.

Preliminary plans.

The first approach was to consider outright purchase of all lands affected by the greatest possible reservoir level. However, since this would be quite expensive and would remove from use lands that would seldom be needed, it was discarded. Another plan was to construct a dam across Cove Creek of sufficient height to shut out backwater from the lake with conduits to pass the Cove Creek flow during low reservoir levels. During high reservoir levels the Cove Creek flow and drainage from Caryville was to be pumped into Norris Lake. The cost of this scheme, approximately $650,000 was prohibitive. Another scheme, also prohibitive in cost, was to relocate Cove Creek, building a dike to protect the town.

Adopted plan.

The plan adopted [9] was to raise the main street of the town to elevation 1,030, to relocate the main highway near the town to elevation 1,035, and to acquire flowage rights over nearly all land within the town below elevation 1,040. To prevent the unsightly conditions during draw-down periods and to provide a lake for recreational purposes a dam was built across Cove Creek about 1 mile below the town to maintain a normal pool level at elevation 1,022.

Elevation 1,040 is 6 feet higher than the top of Norris Dam spillway gates; and, with the proposed operation of Norris Reservoir, it is expected that water will reach this elevation at Caryville very rarely and only at the time of extreme flood conditions. A study during the 50-year record of gaging stations on the Clinch River fails to show any instances where such an elevation would have been reached.

More than 70 structures, including the Caryville public school and the First Baptist Church, were demolished or moved to higher ground. A new street of about 2,000 feet was built near the west side of Bear Creek. State Highway No. 63, which formerly was the main street in the town, and United States Highway No. 25, which adjoined the town, were relocated as a part of the general highway adjustment made by the Authority. The property acquired was purchased in the manner described earlier in this chapter under "Land Acquisition."

Caryville Lake.

The dam for the creation of Caryville Lake is located 1,000 feet below the junction of Cove and Dog Creeks, about 1 mile below Caryville. At the site the drainage area is 36.7 square miles. With water elevation at 1,022 the lake has an area of about 250 acres in the por-

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[9] Draper, Earle S., TVA Replans a Town, The American City, January 1936.

tion immediately adjacent to Caryville. The surrounding land covering a somewhat larger area has been leased to the State of Tennessee for park development purposes, and is discussed further in chapter 8.

Caryville Dam.

The dam is concrete and of the gravity overflow type, composed of an ogee section with short nonoverflow sections on each side. The spillway is designed to discharge 14,000 cubic feet per second with water at elevation 1,027. Transverse joints about 30 feet apart exist between 9 sections. Each joint has a copper water stop near the upstream face, and tongue and groove keys.

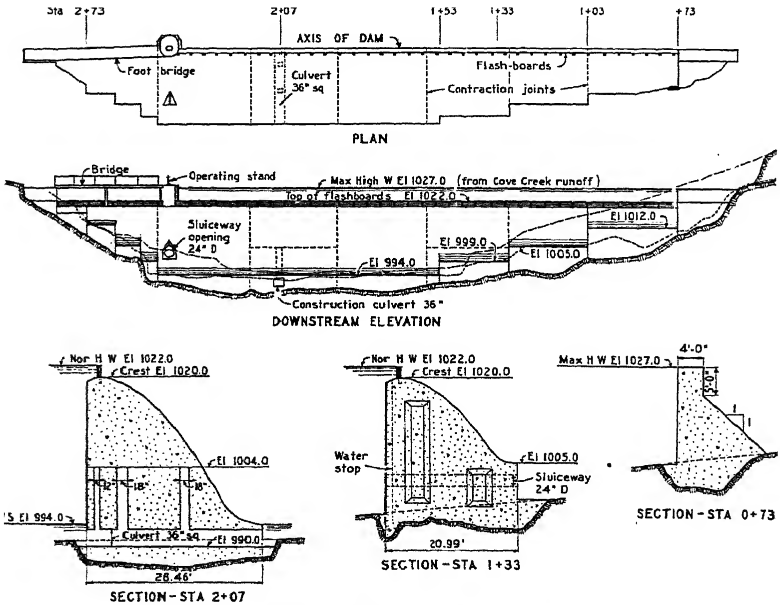

FIGURE 254.—*Plan, elevation, and sections of Caryville Dam.*

A manually controlled discharge conduit for lowering the lake is provided. An elliptically shaped gate tower is built onto the upstream face of the dam to house a 24-inch Chapman sluice gate and to provide for an intake grillage and grooves for stop logs when repairs are necessary to the gate. Two-foot flashboards are provided in four sections to be normally used at the crest of the dam. To prevent the boards from floating away at times of floodwater, they are strung on steel cables anchored to the dam. The pins holding the boards are so spaced that when a flood reaches elevation 1,025, 3 feet above the top of the boards, the sections will progressively raise out, the section at the south end going first.

The dam was constructed by the Authority using force account methods. Letting the job by contract was not advisable because of

the short time available and the necessary close coordination of its construction with the relocation work in and near Caryville. It was authorized October 29, 1935, and was completed on April 21, 1936. A maximum of 156 men, 7 foremen, and 1 superintendent was employed.

A timber trestle was built at elevation 1,022 from bank to bank on the upstream side of the dam and a concrete mixing plant was built on the north bank where the construction plant was laid out.

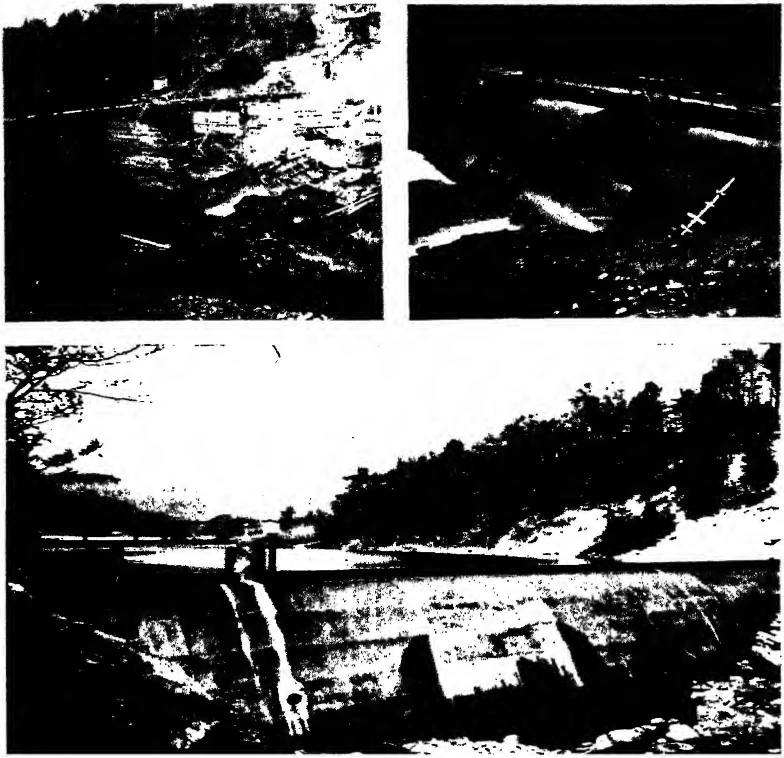

FIGURE 255.—*Caryville Dam—During and after construction.*

Sand and gravel purchased in Knoxville was shipped by rail to Jacksboro, trucked to the job, a distance of about 2½ miles, and dumped into bins on the hillside. It was loaded by cranes into a hopper above the batcher. Cement was delivered in paper bags to the weighing and batching platform, utilizing a chute from the roadway above. Concrete was carried from the mixing plant by a truck operating along the construction trestle. On this truck was mounted an elevated hopper from which concrete was dumped to the forms through chutes to be vibrated in place in the forms.

The stream was diverted through a prepared channel along the south shore by a cofferdam which provided for the excavation of six blocks of the dam. Upon completion of these foundation blocks in the stream bed, the stream was turned through a 36-inch square culvert in the dam and the south section was cofferdammed for excavation, foundation treatment, and concreting.

The construction of the dam necessitated the excavation of 2,910 cubic yards of earth and rock at a direct cost of $8,969.17, the drilling of 465 linear feet of grouting holes, the pouring of 3,587 cubic yards of concrete, and the placing of 3/4-ton of reinforing steel in the outlet tower. The total direct cost of the dam was $60,068.69.

SAFETY ACTIVITIES

Early in 1934 an intensified safety program was organized to promote safety in this vast work spread over such a wide area. The size of reservoir clearance activities, and the potential dangers resulting from the work being conducted in many widely spread areas by small units, required a safety inspector who devoted his full time to this phase of the work.

The table below shows the frequency rate of accidents per million man-hours of exposure and the severity rate of days lost per thousand man-hours of exposure [10] for reservoir clearance operations starting February 1934:

Year ending—	Frequency rate	Severity rate
June 30, 1934	163.4	9.99
June 30, 1935	149.8	6.43
June 30, 1936	74.3	1.52

Working a total of 2,626,714 man-hours, a total of 375 injuries sustained by the reservoir clearance employees was recognized as being of service origin. Of these, eight produced permanent partial disabilities and 367 were temporarily disabling. No fatal accident occurred in clearing the Norris Basin.

For achieving the lowest frequency and severity rates during the year 1936 in a group of five firms of comparable size engaged in logging operations, Norris Reservoir clearance was designated "Honor Roll Company" by the National Safety Council with special citation for having shown the greatest reduction in accident experience for the period 1934–36.

The frequency and severity rates for miscellaneous construction activities within the reservoir, starting in October 1933, are given below:

Year ending—	Frequency rate	Severity rate
June 30, 1934	35.7	3.19
June 30, 1935	62.5	11.17
June 30, 1936	43.9	2.57
June 30, 1937	16.8	.25

[10] Weighted in accordance with standards of International Association of Industrial Accident Boards and Commissions.

The miscellaneous construction activities developed an exposure of 3,921,923 man-hours during which 175 injuries were recognized as being of service origin. One fatality occurred when an employee was burned while trying to start a fire in a small field office stove with gasoline. There were no permanent total disabilities, but 6 of the injuries resulted in permanent partial disability.

Figure 244 shows one of the two mobile first-aid stations used in the Norris Reservoir during the construction period.

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Chapter 8

INITIAL OPERATION AND RELATED DEVELOPMENT

Project operations, while relating primarily to the actual operation and maintenance of the dam, reservoir, and power plant required to fulfill the general objectives of navigation, flood control, and power, also concern numerous other less important problems. Among the latter are such items as fish and game, forestry activities, land management, and recreation facilities such as parks, boating, and swimming.

NAVIGATION, FLOOD CONTROL, AND POWER OPERATIONS

The present method of operating Norris Reservoir is more or less tentative in nature, since the entire system of dams to be built by the Authority is incomplete. Details of the operation probably will be changed as the entire system nears completion and will be modified further as experience dictates.

The Norris project has been in operation since March 4, 1936, when the gates were initially closed and storage of floodwater was begun. However, the operations of the project during this period should not be considered as normal operations as they were adjusted from time to time to assist the downstream construction operations and, because all of the water control projects of the Authority had not been completed, were further modified to assist river traffic by special navigation releases and to assist the other projects in carrying the Authority's electric power load. The reservoir has been used for all purposes during three full flood seasons—those in the winter and spring of the years 1937, 1938, and 1939—and part of one flood season—that of the spring of 1936 immediately following the closure of the dam. Operations have also covered the three low water seasons during the summer and fall of the years 1936, 1937, and 1938.

Since the date the reservoir first began storing water, a number of floods have occurred, including one in the spring of 1936 soon after the dam was closed. During the floods of January and February 1937 storage in this reservoir reduced crest stages four different times at Chattanooga and other downstream points by amounts varying from about 3 to 5 feet, thus preventing considerable damage. These operations also were beneficial in protecting the cofferdams at Chickamauga, Guntersville, and Pickwick Landing Dams. Similar flood control benefits have accrued since 1937.

Likewise, during the three seasons of low flow since the reservoir was first utilized in 1936, water stored in time of floods has been used to supplement natural flows for the benefit of navigation. Maintenance of suitable minimum flows and depths on the lower river has required the release of several thousand cubic feet per second for con-

siderable periods and has made navigation possible at times when otherwise it would not have been.

Power generation at Norris has naturally been at a maximum during the summer and fall when the storage in the reservoir was released to augment the low flow in the main river. During the winter and spring seasons when water at Norris must be caught and held to fill the reservoir, power generation at this plant has been at a minimum; in these seasons when ample water for supplying the Authority's system power demands has been available at the main river dams, the Norris generators have been run as condensers to provide voltage regulation.

The first of the two generators, each of which has a rated capacity of 50,400 kilowatts, was placed in service at 10:30 p. m. on July 28, 1936, and the second went into operation at 2: 06 p. m. on September 30 of the same year.

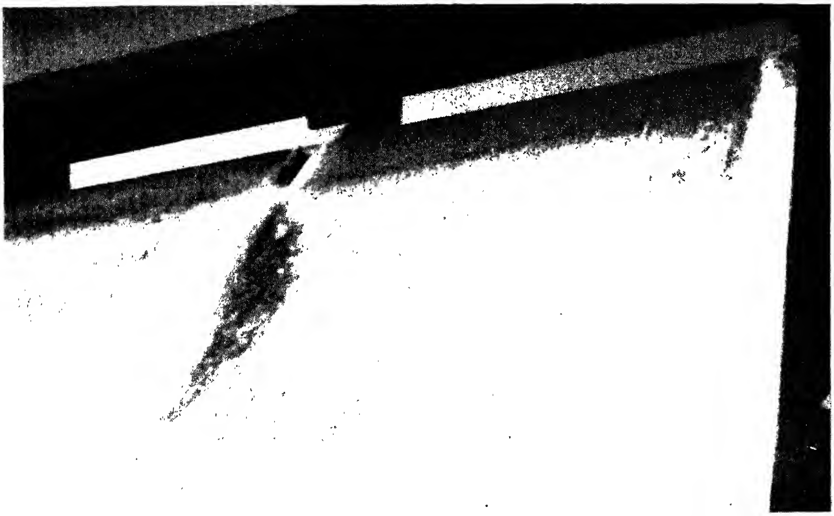

FIGURE 256.—*Release of water stored during January-February 1937 flood. Discharge 30,000 c. f. s.*

The annual gross generation of the Norris power plant from its date of initial operation through December 1938 has been as follows:

TABLE 97.—*Plant generation service data*

Year	Annual output		
	Millions of kilowatt-hours	Average kilowatts	Peak kilowatts
1936 (6 months)	85.33	19,500	66,000
1937	89.55	10,200	51,000
1938	278.25	31,700	93,000

The plant load factor was low during these first 3 years of operation, but it may be expected to improve materially with the building up of electric load on the TVA system.

PRESSURES REQUIRED TO RAISE AND LOWER SLIDE GATES

GATE OPENING	GATE NO. 1-D				GATE NO. 2-D				GATE NO. 3-D				GATE NO. 4-D			
	EL 885 0		EL 1025 0		EL 885 0		EL 1025 0		EL 885 0		EL 1025 0		EL 885 0		EL 1025 0	
	LBS PER SQ IN				LBS PER SQ IN				LBS PER SQ IN				LBS PER SQ IN			
	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER
10	175	40			[illegible]	[illegible]	300	[illegible]	25	60			[illegible]	65	[illegible]	
9.6			320								200					
9	160	135	390		180	105	420	[illegible]	125	50	250		120	65	350	
8	185	130	475		180	105	540	[illegible]	135	50	330		130	50	450	
7	195	130	575		190	130	680	380	142	50	420		135	50	560	
6	200	130	620		195	140	[illegible]	400	145	50	490		145	50	570	
5	200	130	700		200	140	740	400	155	50	600		155	55	650	
4	220	135	750		200	140	720	380	160	60	680		160	60	690	
3	230	135	830		220	140	810	430	160	65	800		170	75	730	
2	245	145	910		230	150	880	480	180	70	930		185	95	850	
1	260	160	1110		240	160	930	580	195	100	1020		200	120	920	
0	280	700	1100		275	700	1000	620	200	700	1050		220	700	1060	

GATE OPENING	GATE NO. 5-D				GATE NO. 6-D				GATE NO. 7-D				GATE NO. 8-D			
	EL 885 0		EL 1025 0		EL 885 0		EL 1025 0		EL 885 0		EL 1025 0		EL 885 0		EL 1025 0	
	LBS PER SQ IN				LBS PER SQ IN				LBS PER SQ IN				LBS PER SQ IN			
	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER	TO RAISE	TO LOWER
10	[illegible]	60	60		105	75			100	40	170	80	85	45		
9.6							200								190	
9	120	60	290		115	55	245		95	30	240	110	95	30	225	
8	125	60	380		115	45	320		105	35	310	150	95	20	260	
7	135	45	490		120	45	440		110	35	400	220	95	35	350	
6	145	55	550		125	60	460		115	35	470	280	100	40	360	
5	150	60	610		140	60	570		125	40	590	320	115	40	480	
4	160	65	660		145	70	630		135	45	680	370	120	40	540	
3	165	70	740		155	75	710		145	50	770	440	125	40	610	
2	175	75	810		170	90	750		160	65	890	490	140	60	690	
1	185	100	910		185	115	825		175	95	1000	590	145	70	780	
0	190	700	1020		200	700	1100		205	700	1020	680	170	700	960	

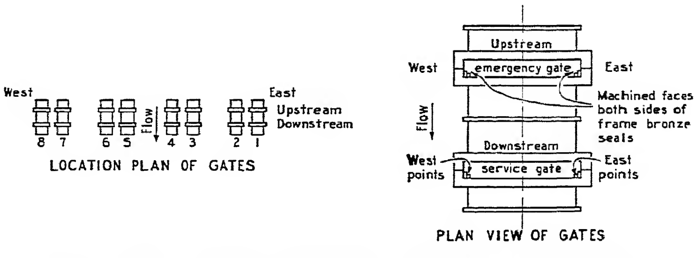

FIGURE 257.—*Pressures required to raise and lower slide gates.*

As opportunities are available over a period of years, the actual discharge over the spillway for various gate elevations will be calibrated in order to check the operating curves obtained from computations and from model tests.[1] Preliminary studies indicate a rather close agreement between actual and predicted discharges. In

[1] See appendix D.

a similar manner the discharge rates of the discharge outlets will be calibrated and operating curves prepared.

Tests were made on the operating equipment for the outlet slide gates at several elevations of the reservoir to determine the pressures required at the pumps to raise and lower each gate. The results of these tests are shown in figure 257. As was expected, the pressures were greatest when the seal was broken prior to the raising operation, but in no case did the actual pressure exceed the design pressure of the pump.

The personnel required for the operation of Norris Dam and powerhouse is shown on the organization chart in figure 258. In addition to the 35 employees listed, guides are provided to conduct visitors through the plant. The two apprentice generating plant operators are in training for work on one of the various projects of the Authority.

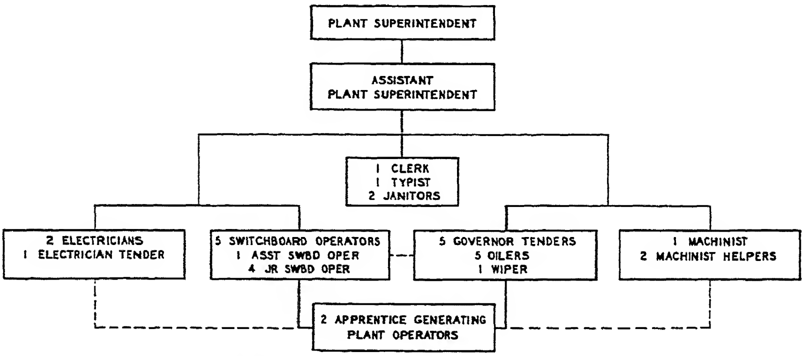

FIGURE 258.—*Organization chart for dam and power plant operation.*

VALUE OF NORRIS RELEASES AT WHEELER AND WILSON

The States of Tennessee and Alabama are annually allotted sums equivalent to a tax on the power produced at the Tennessee Valley Authority dams lying within their boundaries. Whenever water is released from Norris Dam in excess of the natural flow of the river and this additional water is used at Wheeler and Wilson Dams to generate power, the tax on the revenue from this power is divided equally between the States of Alabama and Tennessee. A determination of the amount of water upon which the dividend revenue shall be based is made monthly.

CALIBRATION OF GATES AND SPILLWAYS

MAINTENANCE OF STRUCTURES

INSPECTION OF UNITS

During April 1937 both units were inspected by members of the operating staff and engineering and design organizations of the Authority to determine visually the condition of the runners and

other parts in regard to cavitation and pitting. Runner clearances were also checked at this time. Inspection was made from the scroll cases through the guide vanes and wicket gates and also from a platform suspended in the draft tube below the runner. At the time of the inspection the east unit had operated 1,475 hours, while the west unit had operated a total of 2,444 hours.

On the east unit no signs of active pitting were evident, although a possibility of pitting at a later date was indicated in several places where the paint had been removed. The paint had been removed along the discharge edge of the runner blades, on the inlet edge near where the blades join band and crown, and at one point on the runner band below the blades. Paint had also been removed from the runner blades just below the bars which had been welded between blades to eliminate the vibration which had developed.

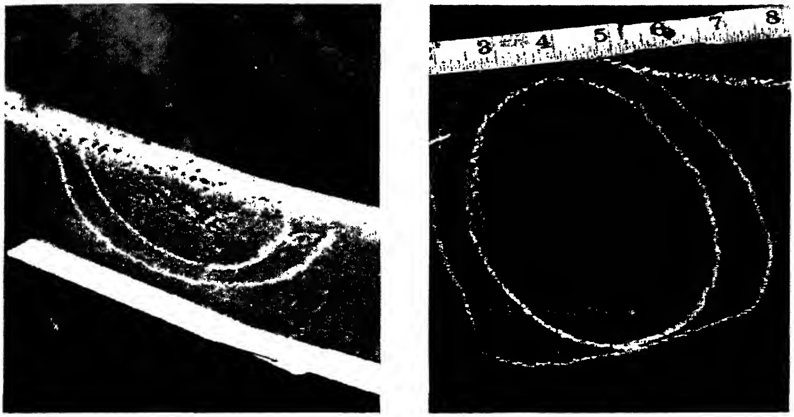

FIGURE 259.—*Cavitation in the west unit.*

In general, on the west unit the same areas as on the east unit had been denuded of paint. In addition, there were numerous areas of very slight active pitting and cavitation—all being near the discharge edge of the runner blades. Active pitting was found in five places where foundry faults had been patched by welding. In three places pitting was apparent in the parent metal, and in at least two other areas this action was evident partly in weld metal and partly in parent metal. Figure 259 shows photographs of typical cavitation in the west unit runner. It was apparent that the runner of the west unit was an inferior casting, and welding of the eroded points was done by the turbine contractor just prior to final acceptance of the units in October 1937.

It was found during the inspection of the units that the paint had been removed and slight pitting of draft tube liner was developing at a spot below and to the right of each rivet head where the draft tube liner connected to the speed ring. It was also discovered that all of the rivet heads were driven so that the edge of the head to the right was not in contact with the surface of the liner. This being

the case, the removal of the paint at these points was much more pronounced, which seemed to indicate that this condition would have a greater tendency toward future pitting. In order to eliminate the possibility of cavitation, the rivet heads were ground down to come in contact with the metal of the liner. In the speed rings, stay vanes, and the spiral casing, no sign of pitting or erosion around rivet heads or lapped joints was evident.

INSPECTION OF DISCHARGE CONDUITS

During April 1937 the portion of the outlet tubes downstream from the emergency gate was inspected. It was found that the concrete downstream from the semisteel liners was eroded in several places, especially where the surface had been patched or some irregularity existed. It was especially evident that formed concrete surfaces were much more resistant to cavitation and pitting than were surfaces that had been finished by troweling. Pitting was also noticed at a few points in the semisteel lining of the tubes.

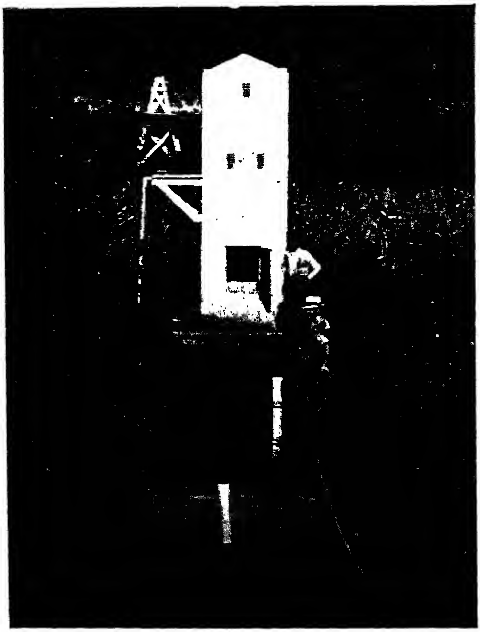

FIGURE 260.—*Stream gages in the Norris Basin.*

The condition of the discharge tubes upstream from the gates was checked in January 1938 by a diver lowered through the opening in the top of the trashrack structure at a time when the gates were closed. The diver inspected No. 6 outlet tube only and found the concrete and metal parts of the bellmouth and tube to be in excellent condition, free from any active erosion or pitting.

STREAM GAGING

The Tennessee Valley Authority has a cooperative agreement with the United States Geological Survey whereby the latter organization operates river gages and furnishes streamflow records to the Authority. Representatives of the Authority inspect recording gages and make check readings of staff gages.

In the Clinch Basin above Norris Dam, 11 recording gages are in service. The construction of the gage houses is similar for most stations; a typical gage is illustrated in figure 260. The most recently constructed gage is situated about 2,000 feet below Norris Dam and furnishes data used in the operation of the dam.

GROUND WATER STUDIES

In order to observe the degree of tightness of the reservoir rim, periodic checks are made on the elevation of the water table in the terrain adjacent to the dam through open core drill holes. The amount of leakage is also recorded from core drill holes under the dam terminating in the lower gallery. Readings are also made on the discharge of springs and the elevation of wells throughout the area. Records made prior to the construction period are available and serve as a direct check on leakage through the reservoir rim.

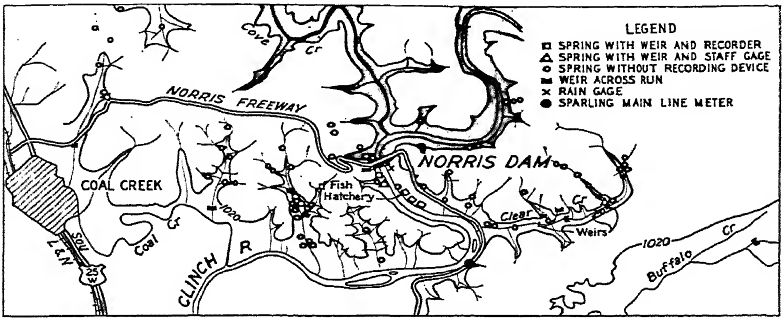

FIGURE 261.—*Location of springs and runs.*

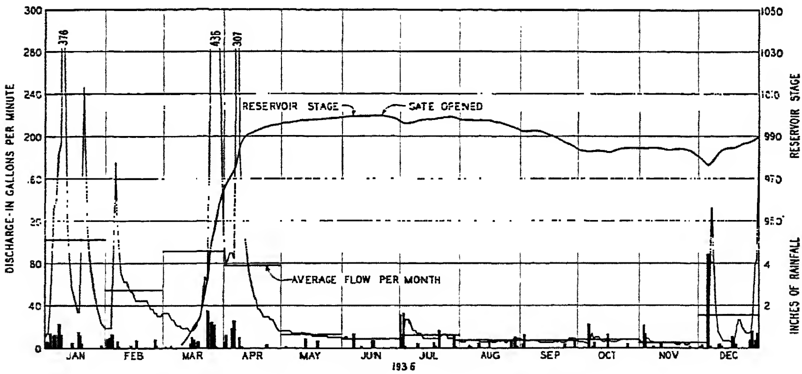

FIGURE 262.—*Discharge of springs and runs.*

The geologic consultants who examined the Norris Reservoir area in 1933 expressed the opinion that because of the limestone character of the reservoir foundation it would be more economical to deal with any leakage after it was discovered than to attempt an expensive exploration which might not locate all possible leaks and yet would extend over much unnecessary ground. Accordingly, a detailed study of ground water discharges in the area below Norris Reser-

voir has been carried on by the Authority. Weirs were placed on springs and runs, and a continuous record was kept of their discharges, so that any flow occurring which could not be attributed to rainfall or other known sources could be detected and investigated. The location of such springs and runs is shown by the map in figure 261, which indicates the widespread area that is kept under observation. The information obtained is presented in the form shown in figure 262, which affords an immediate comparison of reservoir elevation, rainfall, and discharge from the springs. This study gives a comparison of flows in springs and runs after the reservoir was filled with flows occurring during natural river stages and furnishes a reliable check upon the tightness of the reservoir.

EVAPORATION STUDIES

The Authority has maintained an evaporation station about 2 miles below the dam since October 23, 1934, to measure the amount of evaporation and to secure the data necessary to correlate these meas-

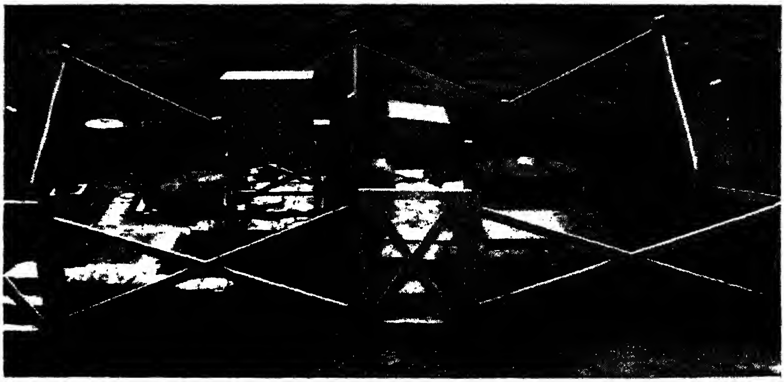

FIGURE 263.—*Norris evaporation station.*

urements with observed meteorological phenomena. In addition to the usual equipment to be found at an evaporation station, this installation includes an automatic weighing device that provides a continuous record of the water evaporated during the day.

SILT INVESTIGATIONS

In badly eroded areas the deposition of silt above a dam is one of the chief factors in shortening the economic life of a reservoir. The Tennessee Valley Authority recognized that a determination should be made of the amount of silt carried by the rivers and of the probable amount of deposition in the various reservoirs.

Studies were initiated early in 1934 to determine data for estimating the life of the reservoir, to indicate where protective measures might be taken, to furnish a measure of the effectiveness of soil erosion control activities and of various forms of vegetal cover, to obtain data

for estimating the clarity of water at probable recreational sites in connection with the recreational development of the reservoirs, and to determine the effect on navigation channels downstream and the probable amount of dredging required.

Extent of field investigations.

During the construction of Norris Dam, silt samples were collected from stations on the Powell River near Agee and Arthur and from the Clinch River at Speers Ferry, Tazewell, Loyston, and at Coal Creek below the dam. Upon completion of the dam the stations at Agee and Loyston were drowned out and abandoned. Samples were also collected from White, Big Barren, and Big Sycamore Creeks as part of an intensive study to determine the run-off and silt characteristics for these three small drainages selected as being typical of the local inflow into the reservoir.

Before the reservoir was completed, about 500 silt ranges were selected in the reservoir, and cross sections were prepared for comparison in the future with cross sections taken along the same ranges after a sufficient time has elapsed for the accumulation of silt.

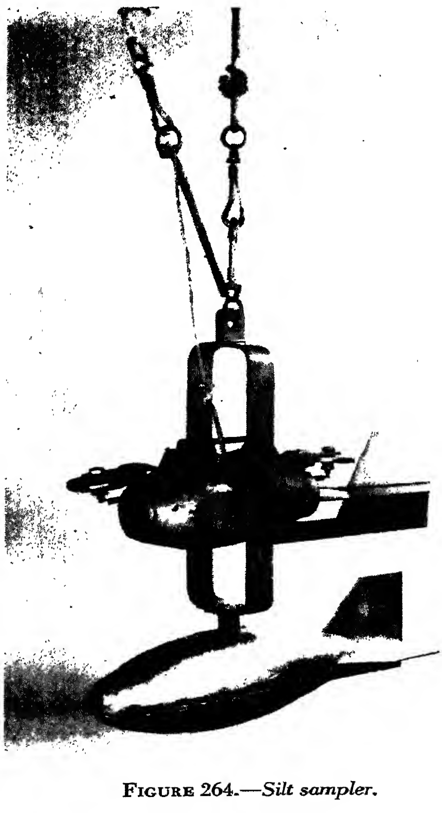

FIGURE 264.—*Silt sampler.*

Manner of collecting silt samples.

During the course of the silt investigations, numerous attempts were made to correlate the silt load of a stream with either the velocity or the discharge of the river. These studies disclosed that regardless of the transporting power of a stream, which is dependent upon velocity and discharge, the stream can carry no more silt than is available. A river at flood stage might carry but little more silt than at ordinary stages, depending upon the condition of the ground. The only way in which this amount can be determined is by periodic sampling.

Present sampling practice requires the use of a streamlined trap open at both ends with a means of closing both ends simultaneously at any desired depth in the river. This sampler, shown in figure 264, was developed by the Authority's engineers. A pull on the trip cord snaps both ends shut, trapping a sample of water. The sample is poured into a pint bottle and transported to the laboratory at Norris for analysis. Samples are generally collected at top, mid-depth, and bottom from several vertical sections, the number depending upon the width and discharge of the stream.

Results.

Results of laboratory analyses are then correlated with the stream discharge to determine the number of acre-feet of silt transported annually by the stream. These studies indicate that the dead storage capacity of Norris Reservoir will not be completely filled by silt in less than 600 years.

MALARIA CONTROL

As an aid in the control of malaria mosquito breeding, a minimum draw-down in the level of the reservoir of at least one-half foot per month is maintained from May through September. While this

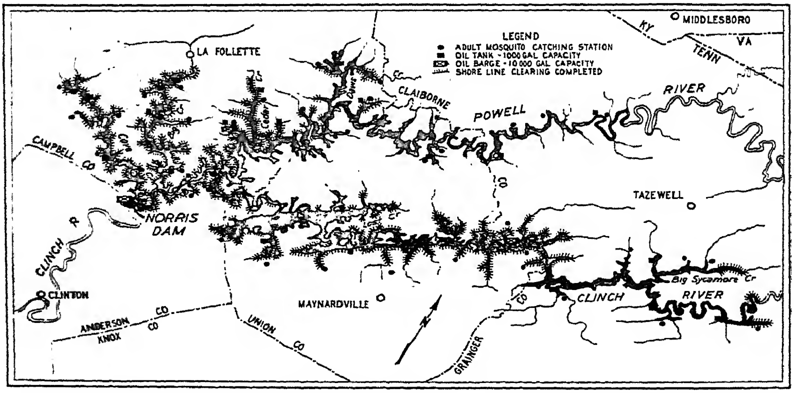

FIGURE 265.—*Location of larvicidal oil tanks and mosquito catching stations.*

measure effectively controls mosquito breeding on the reservoir proper, equipment is provided for larvicidal measures if these should be necessary. Equipment provided for control by larvicidal measures includes 24 oil supply tanks filled with larvicidal oil located at strategic points about the reservoir and 3 inboard motor boats equipped with gasoline-driven pumps for pumping a mixture of oil and water. Two 70-gallon tanks in each boat serve as reservoirs for larvicidal oil. Oil spray boats normally operate with a 2-man crew and can distribute 150 to 200 gallons of oil per day. A 16-foot launch is used for inspection of control work.

During the first few years of reservoir operation practically the only larvicidal work necessary in the Norris area was on the constant

level pools such as at Cove Lake at Caryville, Big Ridge Lake, and the fish rearing ponds located about the reservoir. However, it was necessary at this time to begin a shore line clearing program in order to remove growth that had appeared since the completion of the reservoir clearing operations. This work also included the removal of drift that had become stranded along the shore line and presented breeding places for mosquitoes.

Drift removal and shore line clearance has required between 3½ and 6 man-days per mile of shore line. These figures covered the initial program required after initial flooding of the reservoir, and a reduction of work should be effected upon completion of the initial work.

Inspections for mosquito larvae are made along the shore line to determine the extent of mosquito breeding and the need, if any, for application of larvicides or other control measures. Mosquito catching stations are maintained at numerous points near the reservoir and serve to check the effectiveness of the malaria control program.

Since mosquito work is highly seasonal, a program has been developed so that the employees in the forestry work will have more nearly year-round employment. In this program an experienced crew is maintained working on mosquito control from May to September and on fire control and similar work for the rest of the year.

RELATED DEVELOPMENT ACTIVITIES

RESERVOIR PROPERTY MANAGEMENT

In the management of property acquired for reservoir purposes, due regard is given to the fact that these are public properties acquired in the name of the United States for the purposes stated in the TVA Act, and all plans and programs for the use of the properties are directed toward carrying out these purposes. Consideration is given to the effects of the administration on adjacent properties, in order that the purposes of the act may be most effectively furthered. All health and safety regulations of the Authority and of the several political subdivisions within which these properties are located are observed. Properties are maintained so that physical deterioration is reduced to a practical minimum and so that the physical state of the property is maintained at as high a level as is consistent with the state of the property acquired and a reasonable expenditure of funds. In addition to these general policies, reservoir property is maintained in such a manner that soil erosion is reduced to a point consistent with the resulting increase in the effective life of the dam and reservoir. The property is also administered in such a way that on a continuing basis operating revenues will equal operating expenses, except for expenses incurred in:

1. Administering malaria control measures.
2. Administering property used for experiments and demonstrations as authorized by section 22 of the TVA Act. (See appendix J.)
3. Providing police and guard service for the dam and hydroelectric plant.
4. Protecting the general public visiting the dam and the hydroelectric plant.

Rates and fees for the use of the property are designed to secure the maximum use consistent with the foregoing policies.

In order to aid in properly managing the property, a land use plan, which is in effect an inventory of possibilities, is being prepared. This plan will be used as a background or base map outlining certain basic possibilities. It will indicate:

1. Areas which are not now in forests, which are not needed for public or semipublic recreation areas, and which, by topography and composition are suitable for agricultural purposes.
2. Areas which, by reasons of such factors as location, accessibility, and scenic resources, are suitable for public recreation development.
3. Areas which, by reason of location, accessibility, and other factors are suitable for semi-public recreation development.
4. Areas set aside as demonstration and experimental areas.
5. Areas now in forest and to be reforested which, in the aggregate, are to be devoted to the maximum production of forest products on a continuing annual basis.
6. Areas not designated for any of the above uses, which can be considered for private use when the demand requires such use.

EROSION CONTROL AND REFORESTATION

The control of soil erosion on the protective strip to be acquired around the reservoir was recognized as a significant problem by the Authority early in 1933. It was evident that a large percentage of this strip would have to be reforested to insure maximum usefulness. Accordingly, a number of Civilian Conservation Corps camps were located on or adjacent to these lands in the fall of 1933. The camps have been continued in varying numbers since. In 1938 three camps were available for work on the area.

The TVA-C. C. C. camps are operated jointly by the Authority and the United States Forest Service. The Authority furnishes the technical supervision of the program; actual supervision of work on the job and the actual operations of the camps are handled by the United States Forest Service. The camps are available for general watershed protection work, but special emphasis is placed on the control of erosion and the reforestation of the reservoir lands.

At the time this program was initiated, there was little information available as to methods and technique of erosion control suited to this region. It was therefore necessary to devote a great deal of time and energy to the development of erosion control technique that would be practical for use on the Norris area as well as on other reservoir and private lands. During the earlier years of this program, the Norris Reservoir lands were used as a testing ground for erosion control and reforestation measures which have since been applied to other areas in the valley.

During the first 3 fiscal years a high percentage of time of the C. C. C. camps was devoted to the construction of the engineering structures and installation of water control measures on the more critically eroded areas. During the fiscal years 1936–38 the operations consisted largely of reforesting areas in need of planting where the erosion was not far enough advanced to require intensive engineering treatment prior to planting.

In order to provide planting stock for this area and to provide for the needs of other reservoir and private lands the Authority established a forest tree nursery at Clinton, Tenn. This nursery occupies 147 acres and has an annual production capacity of approximately 12,000,000 seedlings.

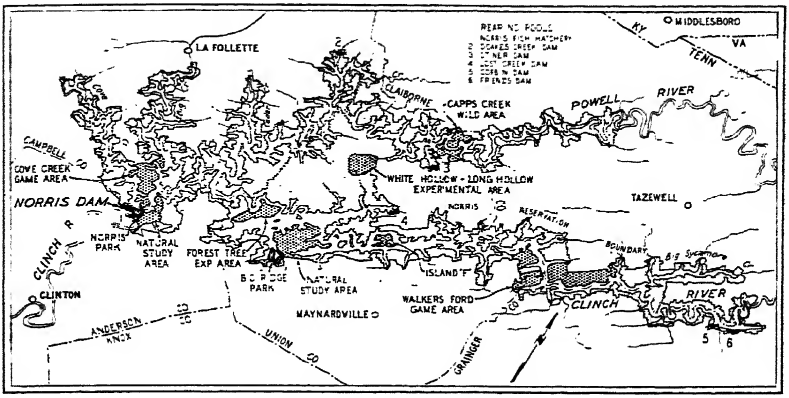

FIGURE 266.—*Special land uses.*

TABLE 98.—*Summary of tree planting activities*

Fiscal year	Number of tracts	Number of trees planted: Black locust	Pine	Others	Total trees	Black walnut seed	Total area, acres	Man-days [1]
1934	61	667,000	21,400	55,900	744,300	3,250	322	1,495
1935	158	1,351,500	373,200	37,400	1,762,100	100,050	924	3,724
1936	161	173,400	1,031,000	143,200	1,347,600	99,100	2,530	2,893
1936 (experimental forest)	1	17,500	2,000	2,500	22,000	2,500	25	49
1937	74	127,600	3,724,600	58,900	3,911,100	3,620	[2] 2,512	7,829
1938	135	624,500	3,422,800	64,900	4,112,200	[3] 83,420	3,656	8,390
Totals to date	530	2,961,400	8,575,000	362,800	11,899,300	291,940	9,969	24,380

[1] C. C. C. labor.
[2] 645 acres replanted not included in this total.
[3] 24,000 mixed acorn spots included in this figure.

TABLE 99.—*Summary of soil erosion control*

Fiscal year	Number of tracts	Tract area, acres	Project area, acres	Number of masonry check dams	Number of temporary check dams	Bank protection, square yards	Man-days [1]
1934	125	12,362	537	1,853	7,151	288,356	56,784
1935	259	25,355	1,190	1,066	5,913	1,817,562	91,402
1936	18	1,013	52	17	60	81,385	3,868
Grand totals all years [2]	402	38,730	1 779	2,936	13,124	2,187,303	152,054

[1] C. C. C. labor.
[2] Town of Norris not included.

FOREST MANAGEMENT

The Authority is gathering data essential to the development of a forest management program. Such a program will provide for the protection of the forests from fires and insects, the improvement and development of the timber stands so that their maximum production may be realized as soon as possible, a year-by-year harvest of that

FIGURE 267.—*An example of erosion control. A. Deeply gullied area before control was started in December 1933. B. Same area in July 1935 after 2 years' growth of locust.*

portion of the annual growth not needed to increase the growing stock or forest principal, and the cutting and marketing of forest products.

There have been set aside 10 forest experimental and demonstration areas totaling 5,586 acres for the purpose of conducting certain forestry experiments and demonstrating various types of forestry practices. Cooperative studies are being conducted in the White Hollow watershed to determine the changes in vegetative cover on lands under the Authority's administration and the effect these changes have on the surface flow of water and soil erosion. A cooperative planting

project is under way to determine the tree species and mixtures best adapted for watershed protection and financial returns in this area. Forest stand improvement work has been undertaken in a few demonstration areas to determine the effects on the stand and rate of growth. Wherever possible, this work is conducted in cooperation with other existing agencies.

FISH AND GAME

The fish and game management programs are carried out in cooperation with the United States Bureau of Fisheries, the United States Bureau of Biological Survey, and with the assistance and cooperation of the Game and Fish Division of Tennessee. Under this program the activities fall into two main headings: (1) The development and management of fisheries and fish life, and (2) the development and management of game, including therein both birds and mammals. Fishery activities include all phases of work dealing with the development and maintenance of the aquatic resources. Game management activities include the establishment of game management areas, refuge areas, wild life sanctuaries, and other restricted localities in which quail, grouse, wild turkey, deer, and economically important furbearing forms may be increased.

Fish.

After the first few years of existence, impounded reservoirs ordinarily become decidedly lacking in fish productivity, primarily because of the large annual fluctuation in the level of the water in the reservoir which prevents the establishment of aquatic vegetation in the shallow water along the shore and also prevents fish from propagating under natural conditions sufficiently to maintain the fish population. In anticipation of this problem the Authority constructed a fish hatchery just below Norris Dam where fish, particularly large mouth bass, will be hatched for the purpose of stocking Norris Reservoir. This hatchery consists of three 2-acre fish ponds and is in the charge of a fish expert from the United States Bureau of Fisheries. Following the hatching of the fish, the fry are collected and transferred to a series of rearing pools located around Norris Lake. These rearing pools consist of small dams across strategic points of small incoming streams, behind which constant level ponds are established under controlled conditions. Five such rearing ponds were completed by 1938. In these ponds the fry are retained until reaching a size sufficient to insure their safety upon transplantation into the lake. Figure 266 shows the location of the six rearing pools.

Although each of the fish dams differs in size and over-all dimensions, their design and construction are similar. The Doakes Creek Dam is typical. It was built by Civilian Conservation Corps labor under the supervision of the Authority and consists of a rolled earth fill dam with a top elevation of 1,027 and an over-all length of 750 feet. It has a maximum height of 30 feet, a maximum base width of 165 feet, and a top width of 10 feet. Both faces have a 3:1 slope and are covered with stone riprap since at extreme high water in the reservoir the entire structure may be submerged. A spillway 150 feet long, located at the west end of the dam, is paved with riprap

and can handle a flow of 1,500 cubic feet per second, which is the estimated maximum run-off from the 2 square miles of drainage area above the dam. Estimated quantities involved in this construction are:

Embankment	cubic yards	26,000
Riprap	square yards	9,200
Paving	do	2,300
Masonry	cubic yards	300
Concrete	do	100
Reinforcing steel	pounds	9,000

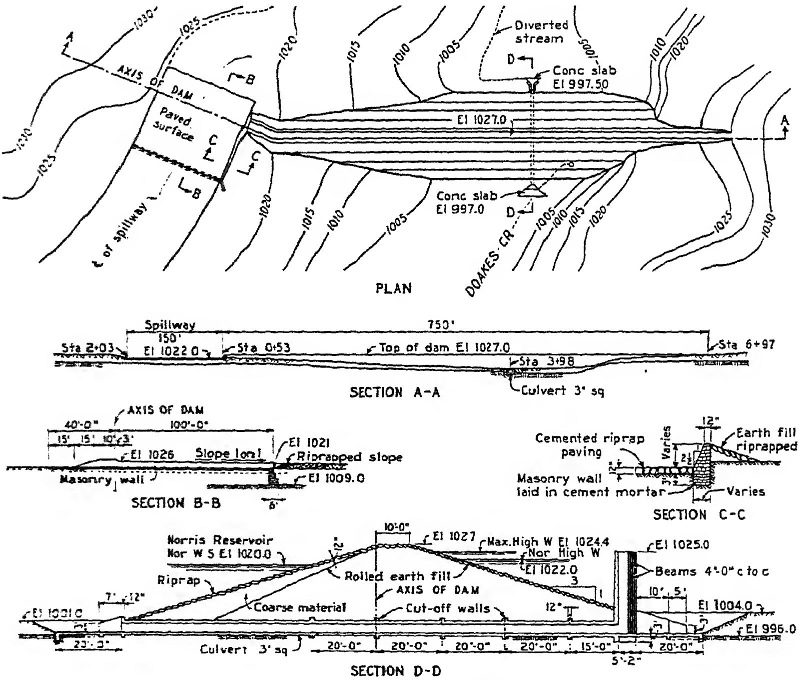

FIGURE 268.—*Doakes Creek Dam.*

The 45-acre constant level Big Ridge Park Lake has been stocked with fish as an adjunct to its recreation potentialities.

All fisheries work must necessarily be based upon an understanding of the water environment involved which includes the chemistry of the water, the cyclic behavior of chemical and thermal factors, and the abundance and distribution of the micro-organisms which are the basis of fish foods. In order that practical, applied fisheries activities may be undertaken upon solid scientific facts, a well-equipped laboratory boat has been placed upon Norris Reservoir, and intensive and extensive chemical, physical, and biological studies are being carried out.

Game.

The U. S. Bureau of Biological Survey and the Tennessee Division of Game and Fish have cooperated in the establishment of various types of game areas. In general the point of view is that of developing the land surrounding the Norris Reservoir in terms of that type of wild life particularly adapted to it. It is not considered advisable to attempt building up any area in game for which that area is not specifically and naturally suited. Thus, the problems become essentially those of rehabilitating the areas under consideration with what in each case was the original native wildlife of that region. The Norris Reservoir lands have been carefully studied and various tracts of land have been allocated for the management and development of game. As a result of the surveys, certain lands have been set aside for the development of quail, others for grouse, and others for turkeys, or for deer, or for various combinations of these species. The use of the areas has been determined by the type of life best suited to them. Exotic forms will not be introduced. Briefly these areas may be classified as game management areas and wildlife sanctuaries, based upon the type of treatment which the game is to receive upon that land.

In the game management areas everything possible is being done to increase game, such as the planning of food, shelter, cover, and protection against both active and passive enemies. Entrance by interested individuals is permitted, but no fires may be built and no mutilation of trees or disturbance of wildlife is allowed. Four areas of this type have been established; the Cove Creek area, for upland game birds and deer, 2,120 acres; the Valley Ford area, for upland game birds and deer, 5,050 acres; the Lonesome Valley area, for upland game birds, 2,625 acres; and the Island F Wilderness area, for the study of the rehabilitation of fauna and flora with minimum disturbances by man, 450 acres.

NAVIGATION ON THE RESERVOIR

Two 60-foot all-steel Diesel-motored boats are in use at present for Norris Reservoir project operations. Figure 269 shows one of these boats which is powered by a 100-horsepower engine, weighs 16,300 pounds, and has a maximum speed of 14.5 miles per hour and a cruising speed of 12.5 miles per hour. They are available for towing service relating to the transportation of timber, stone, fertilizer, and other materials on the reservoir and for the movement of crews engaged in forestry and other maintenance and improvement work. Forty-eight passengers can be accommodated on each boat. During the summer season sightseeing trips are made to various points in the reservoir for the public at times when the boats are not needed for other purposes. A charge is made to defray all operating expenses.

No navigation lock was constructed at Norris, but studies have been made and the details completed for the transfer of coal on Norris Reservoir and through the dam by means of a mechanical handling plant which may be installed in the future. A gallery to allow passage of the coal through the dam has been constructed. This method of handling coal (the principal potential tonnage) is based on actual installations in other parts of the country and could be completed should the volume of traffic be sufficient.

RECREATION

Although Norris Lake is only about 70 miles long, it is so cut by promontories and deep mountain coves that its shore line measures more than 700 miles at elevation 1,020. Since lakes are not common in the Tennessee Valley, people were eager to take advantage of the opportunity for swimming, boating, and fishing. During the summer of 1937, the first after the lake was filled, there were over 600 privately owned boats on the lake. In 1938 this number had increased to approximately 1,200.

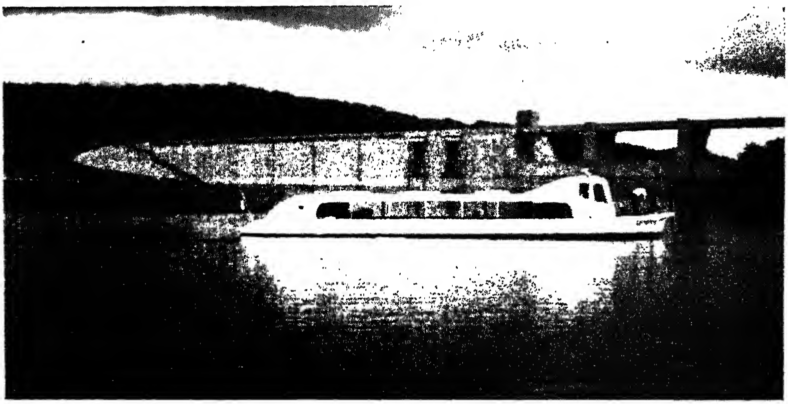

FIGURE 269.—*Sixty-foot all-steel Diesel-motored boat.*

The shore land necessarily purchased by the Tennessee Valley Authority primarily for reservoir protection purposes also affords control over access to the water and control of buildings and other improvements along the shore, and every effort is being made to secure a pleasing and appropriate development.

The Tennessee Valley Authority is not authorized to develop recreation facilities to serve the general public; but recognizing the tremendous demand for recreation facilities on Norris Lake, the Authority has developed plans and made certain demonstrations to guide the unified development, through other channels, of the lake's recreation potentialities. These activities have been carried on under sections 22 and 23 of the act. In this program of recreation planning and development, the Authority has enlisted the cooperation of other agencies, notably the National Park Service and the Civilian Conservation Corps. Two demonstration parks and a boat harbor were developed on the shore of Norris Lake, on land which already had been acquired for reservoir purposes. In addition to the land, the Authority contributed designs, a portion of the materials, some skilled labor, and construction supervision. The National Park Service contributed design supervision and shared in the expense of materials. The Civilian Conservation Corps contributed all unskilled and some skilled labor, supervision of enrollees, and certain details of design. It was expected that these demonstration facilities

would serve as a guide to other agencies and individuals in the design, development, and operation of similar facilities on other parts of Norris Lake and on other lakes. The success of the demonstrations is indicated, in part, by progress in State park development in Tennessee since construction of the Norris Lake facilities was inaugurated. While the parks were under construction, the State of Tennessee did not have a Division of State Parks and had no park system. In 1937 the Tennessee Legislature passed an act setting up a State Department of Conservation, which included a Division of State Parks.

FIGURE 270.—*Norris boat landing.*

This branch of the State government is now actively developing a State park system which will include other parks similar to Norris and Big Ridge. One of those State parks is being built at Caryville on a portion of the land aquired for reservoir purposes. Here again the Authority is preparing plans for the park, although the final approval rests with the State. The land on which the park is located has been leased to the State for a nominal fee, and the State will maintain and operate the park after its completion.

A boat landing has been constructed at the site of the quarry from which concrete aggregate was obtained for the construction of the dam. This development includes boat docks, boathouses, comfort stations, minor repair facilities, provisions for parking automobiles and trailers, a gasoline service station, and other items necessary for the convenience of the large number of pleasure craft using the reservoir. Some idea of the popularity of boating at Norris Reservoir may be obtained from the fact that two years after the completion of the dam approximately 1,200 boats were in use. Figure 270 gives a general view of the boat landing and of the boating activities.

In addition to the quarry boat landing, nine other boat docks at various points on the reservoir were in operation during the summer of 1938. These are all operated by concessionaires. Rates charged for services are established by the Authority and are uniform on the entire lake.

Pending the completion of a coordinated land use plan, no further recreational development has been projected except that studies are being made to determine the feasibility of establishing a Negro recreational area on Norris Lake. As soon as this plan is completed, however, a number of developments may be initiated. Appropriate regulations by the Authority to insure an orderly and well-planned development will be adopted. Rates for the use of facilities and the quality of services in these developments will be regulated by the Authority.

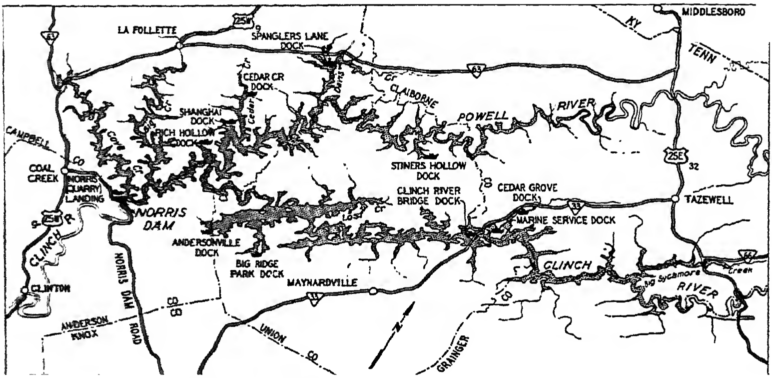

FIGURE 271.—*Boat landings in operation during 1938.*

NORRIS PARK

This park skirts the south shore of the Norris Reservoir for more than 3 miles, its 3,887 acres extending from the lake to the town of Norris. Main features of the park are improved picnic grounds containing parking areas, shelters, toilets, table and bench combinations

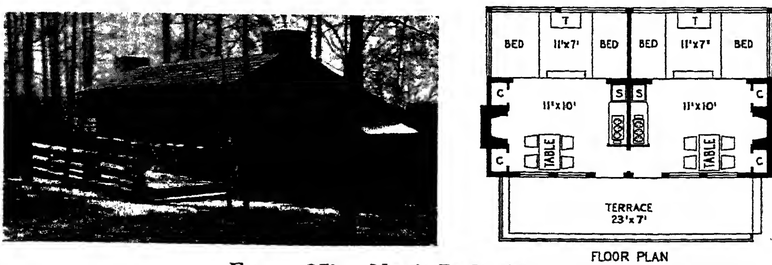

FIGURE 272.—*Norris Park cabin.*

ovens and drinking fountains; a camp ground for trailers or tents, also provided with a centrally located bathhouse, electricity, water supply, tables and benches; and a lodge, which contains a tearoom and a small store carrying food supplies for cabin occupants. In addition, there is an outdoor theatre seating approximately 500 peo-

ple. Twenty cabins are provided, similar to the one shown in figure 272, of which five are the duplex type. All of these cabins are supplied with running water. Twenty units have inside toilets and showers. The remainder are served by a centrally located bathhouse. Each cabin is fully equipped and completely furnished so that vacationers need supply only food. A riding stable was also constructed and is operated during the summer season. A floating dock affords swimming facilities for experienced swimmers. This park has been extremely popular, the annual attendance at the park being close to 100,000. Cabins are in great demand and are generally

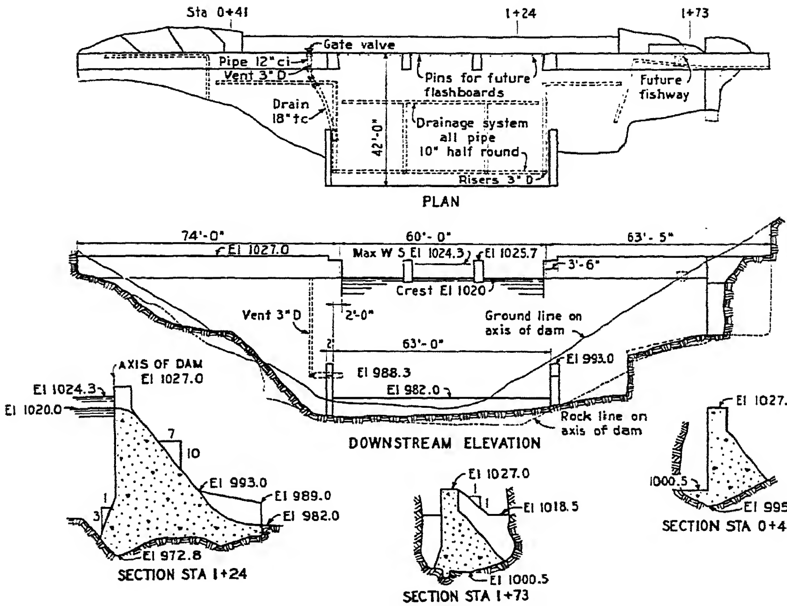

FIGURE 273.—*Big Ridge Dam.*

reserved for several weeks in advance. The park season extends from about April 15 to November 1, depending on weather conditions. The tearoom, store, and riding stables are operated by concessionaires. The balance of the services are provided by the Authority.

BIG RIDGE PARK

Big Ridge Park contains approximately 4,500 acres and is almost entirely surrounded by water. The nature of the topography is very rugged, with heavily wooded broken ridges crossing the area. In order that the seasonal draw-down of the main reservoir would not interfere with swimming and boating at Big Ridge Park, one of the arms of the reservoir was dammed so that the impounded water in this area is not subject to the seasonal draw-down of the reservoir. This dam is about 50 feet high and nearly 200 feet long, with a 60-foot

wide spillway section. It was constructed of concrete, with a timber guard rail providing a footway along the crest.

The area of intensive development in this park consists of about 100 acres of land around the margin of Big Ridge Lake. Nineteen cabins, similar to those built at Norris Park have been constructed to accommodate tourists and summer visitors. Building exteriors at Big Ridge Park are either of rough sawed siding or of logs, sometimes combined with native stone work for variety. The logs were selected from timber cut by the Authority in its reservoir clearance operations.

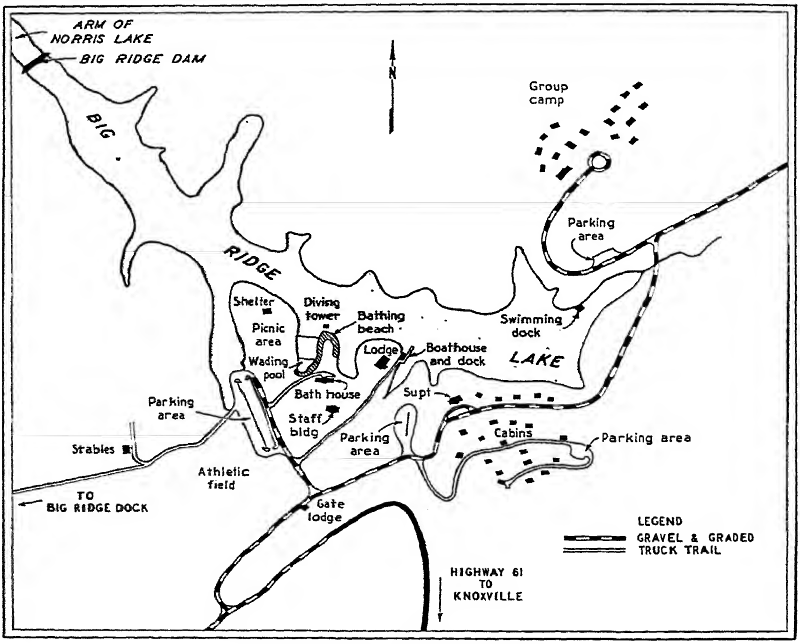

FIGURE 274.—*Big Ridge Park.*

An excellent bathing beach has been located between two of the small arms of Big Ridge Lake. The beach itself consists of a 20-foot wide strip of sand around the shore line of the peninsula, with a grass plot in the middle. A wading pool for children and a beginners' pool, paved with concrete, are separated from the deeper water used by more experienced swimmers. Springboards, floats, and a diving platform are also provided. Water is circulated through the shallow pool by means of a recirculating pump.

In addition to the cabins and bathing facilities, boats and canoes are available at a small boathouse. A small riding stable also forms one of the attractions. Meals and food supplies for campers are available at a centrally located lodge.

A group camp, consisting of 11 cabins, 2 wash houses, 2 bathhouses and a central dining room is also an integral part of the park. This camp is rented as a whole to groups such as the Girl Scouts and 4-H clubs. The riding stable, beach, tearoom, and store are operated by concessionaires. Other services in the park are furnished by the Authority. During the year approximately 50,000 people visited this park. An admission fee of 10 cents per person is charged on Saturdays, Sundays, and holidays. On other days admission is free.

COVE LAKE STATE PARK

A demonstration State park is being developed near the town of Caryville in Campbell County, Tenn. This park contains approximately 1,000 acres, of which 600 acres are TVA-owned land, plus a 210-acre constant level lake, formed by damming the Cove Lake arm of Norris Lake. In addition, the park includes a 93-acre tract donated to the State of Tennessee by the First National Bank of La Follette, Tenn. The TVA lands have been leased to the State of Tennessee through the State Department of Conservation.

The area is being developed by the National Park Service and the Civilian Conservation Corps. All plans are prepared by the Authority and are reviewed and approved by the State of Tennessee before presentation to the National Park Service.

Construction was started in July 1937. The initial improvements include the construction of the main road system, the service group of buildings, and a part of the water supply and distribution systems.

The area is being planned as an intensive use area for the combined use of the local public and the tourists traveling through the park on United States Highway 25W and State Highway 63. A feature of the park will be a group of 20 model tourist cabins, equipped with modern conveniences including steam heat, a restaurant, a pavilion for gatherings, and a superintendent's dwelling. Other facilities planned include picnic area, playgrounds, a swimming pool, and possibly a golf course. The park will be operated and maintained by the Department of Conservation of the State of Tennessee.

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This bibliography includes all known published material concerning the contents of chapter 8. The fact that a publication is included should not be interpreted as giving official endorsement or confirmation of its contents.

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Chapter 9

COSTS

The accounting work was done partially at the field construction office located at the dam and partially in the Knoxville accounting office. Generally speaking, the field office kept all the actual construction and reservoir activity accounts, while all miscellaneous administrative and overhead accounts were kept at Knoxville. Field accounts were kept entirely by hand method, but for the greater part of the job (after late in 1934) all control accounting work was done on a Hollerith machine.

Engineering planning and design accounts and administrative overhead and general engineering service accounts, kept in the central office, followed conventional accounting methods.

A cost engineering staff was employed at the dam for analyzing and interpreting actual construction costs. This staff concerned itself with the construction of the dam, powerhouse, original switchyard, Loyston dike, and highway approaches to the dam, as well as reservoir rim grouting work and a major portion of the site improvement. This was the principal cost engineering work done by and for the project.

Since this was the first large construction project undertaken by the Authority, it was not unusual that changes were made in the accounting and cost methods as the work progressed. The methods finally adopted are set forth in this chapter. Many of these have been adopted on subsequent TVA construction projects, while others have sustained further improvements.

FIELD ORGANIZATION AND RESPONSIBILITIES

The division of duties between the field accounting and cost engineering organizations with respect to the dam construction work was that the accountants were responsible for general accounting and bookkeeping work, while the cost engineers were responsible for allocation of charges to the proper account, and the interpretation and application of cost information.

Accounting staff.

This section was headed by a chief clerk who was responsible administratively to the construction superintendent and functionally to the construction accountant, stationed at Knoxville. The section handled all work concerning accounting, timekeeping, pay rolls, warehouse accounting, collection and reporting of cash, clerical work on shop orders, and the preparation of financial statements and trial balances of the cost accounts. It prepared field vouchers for purchases and contracts and handled interdepartmental transfers. The time office was one of its units, the chief timekeeper being directly re-

sponsible to the chief clerk. The warehouse accounting clerk was also under the functional direction of the chief clerk, although the regular storekeeping operations were carried on under the immediate supervision of a storekeeper responsible to the superintendent.

At first, bookkeeping and accounting work was done for the project in the Knoxville accounting office, with a field office to handle field pay rolls, to requisition and receive materials, and to handle other field cash disbursements. On July 1, 1934, the field accounting office was established and it functioned until work was completed.

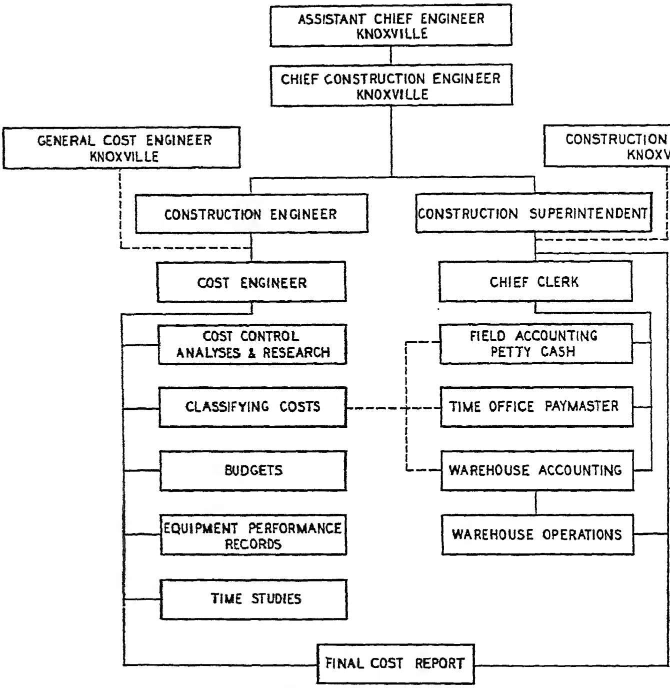


FIGURE 275.—*Accounting and cost engineering organization.*

Accounting work for the camp was done by an independent office not directly connected to the dam organization. Employees' statements were rendered each pay period to the dam accounting office, from which charges were made for camp services such as meals, room rent, and other services. Construction of the camp and the town of Norris was by a TVA organization separate and apart from that which constructed the dam.

Reservoir clearance; cemetery, highway, and railroad relocation; property protection; and other reservoir work was done by separate TVA organizations not responsible to the dam construction organization. However, as previously mentioned, the control accounts for

these reservoir activities were kept at the dam office. Each organization doing work in the reservoir forwarded accounting data to the dam accounting office for entry. Details, such as time records and other supporting data, were kept in the temporary field offices at the site of operations, but the official accounts for all reservoir operations were maintained at the dam.

Cost engineering staff.

This staff was organized early in 1935 under the direction of a cost engineer who was responsible to the assistant construction engineer and who worked in cooperation with the general cost engineer of the Knoxville office. Figure 275 shows the direct and functional lines of responsibility. The duties for this organization included allocation of charges in accordance with the approved classification of accounts. It prepared the bases for the distribution of clearing accounts, applied plant and equipment depreciation schedules, and worked in cooperation with the accounting staff, handling matters where technical questions entered into the accounting operations. In analyzing and interpreting cost data, the cost engineering staff based its work on the cost books and records kept by the accountants. The cost engineers also gathered data for record reporting and performed cost estimating and budget work.

The principal function of the cost engineering staff was to allocate charges; analyze, interpret, and publish current cost data for job control purposes; prepare record analyses of completed feature costs; and prepare estimates and budgetary reports.

Field time studies on certain construction operations were made for limited periods of time to secure more detailed information on these operations. A special field study group of three men was organized for this purpose; and at the end of their duties at Norris, the group was transferred to other construction projects of the Authority where they served in the same capacity.

The cost engineers also collected and recorded detailed records of individual plant and equipment performance and cost. These were needed as an aid to job and cost control and in selecting equipment for future jobs.

CONSTRUCTION ACCOUNTING

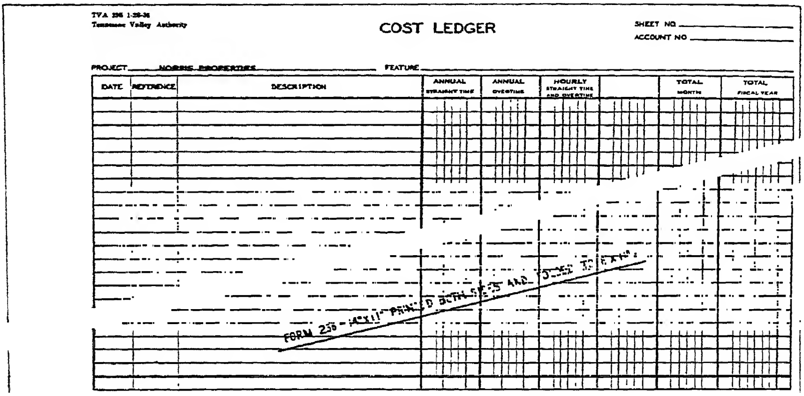
The practices and procedures developed as the work progressed followed closely methods and principles previously developed for construction work. The books of account consisted of a general ledger, voucher register, invoice register, stores ledger, cost ledger (fig. 276), equipment ledger, and warehouse receiving and issuing reports. Entries to the general ledger were made direct from journal vouchers with no journal voucher register being used.

Standard purchase vouchers (fig. 277) were prepared in the Knoxville office, invoices being certified by the field office and entered in the field vouchers payable register and forwarded to the Knoxville office for vouchering and payment with form 1035 (fig. 278). All disbursements, except emergency payments, were made by the treasurer. Emergency payments were made by an agent officer of the treasurer from the small fund established for this purpose. Details of accounts

were kept in the field office books with controls kept by the accounting office in Knoxville.

Classification of accounts.

The classification of accounts used at Norris differs slightly from the master classification of accounts as established in the spring of 1937 for all of the Authority's construction projects. Fundamentally, there were 10 key cost accounts for construction features, in addition to the clearing, equipment, and plant accounts.


COST LEDGER

SHEET NO.
ACCOUNT NO.

PROJECT NORRIS PROPERTIES FEATURE

DATE	REFERENCE	DESCRIPTION	ANNUAL STRAIGHT TIME	ANNUAL OVERTIME	HOURLY STRAIGHT TIME AND OVERTIME		TOTAL MONTH	TOTAL FISCAL YEAR

FIGURE 276.—*Cost ledger.*

The key accounts were:

ND–0 Improvements to station site.
ND–1 Reservoir activities.
ND–2 Freeway.
ND–3 Dam and spillway.
ND–4 Hydraulic structures and equipment.
ND–5 Powerhouse building.
ND–6 Powerhouse electrical equipment.
ND–7 Switchyard and switchyard equipment.
ND–8 Station mechanical and general equipment.
ND–9 Other activities (including reservoir rim treatment and Loyston dike).
NDC Construction clearing account.
NDE Construction equipment.
NDP Construction plant.

Each of these accounts was subdivided into as much detail as circumstances warranted. It was the intention to include in the clearing account all charges for construction equipment, the identity of which could be maintained, the value of which was intrinsic, and which could be expected to have a resale value other than salvage at the end of its period of construction service on this project. On the other hand, the equipment account was to include such structures

and buildings that would be of use only on this project, and at the end of the construction period could be expected to have only a salvage value. The cost of assembled construction plants was not readily available because the plant accounts and equipment account

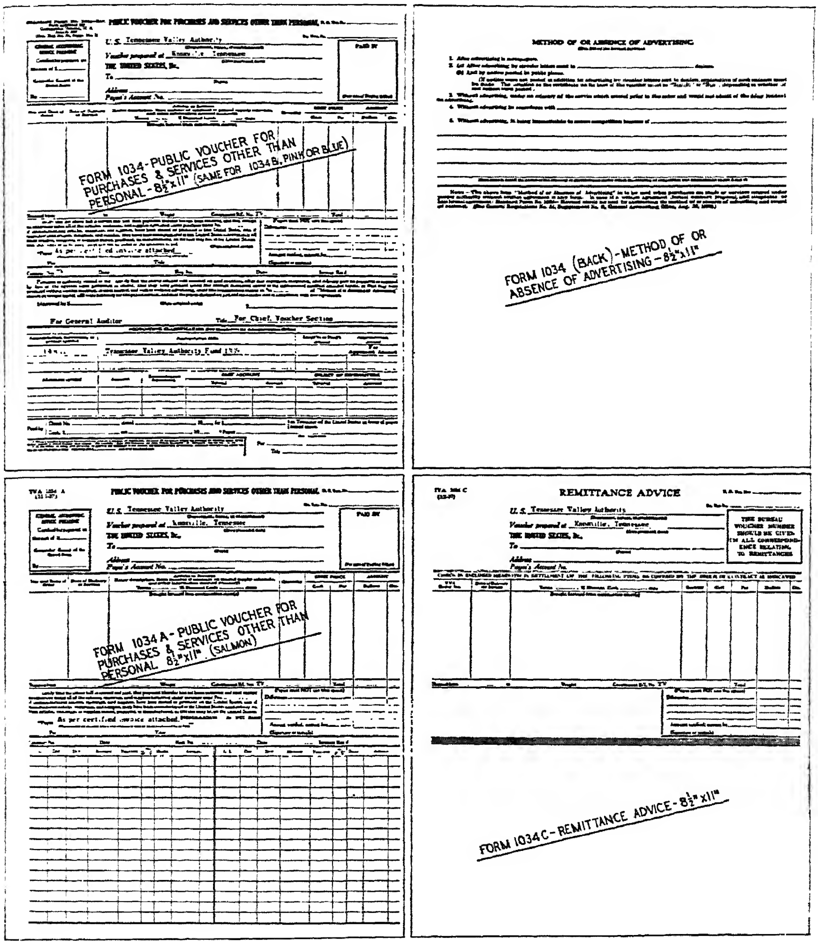
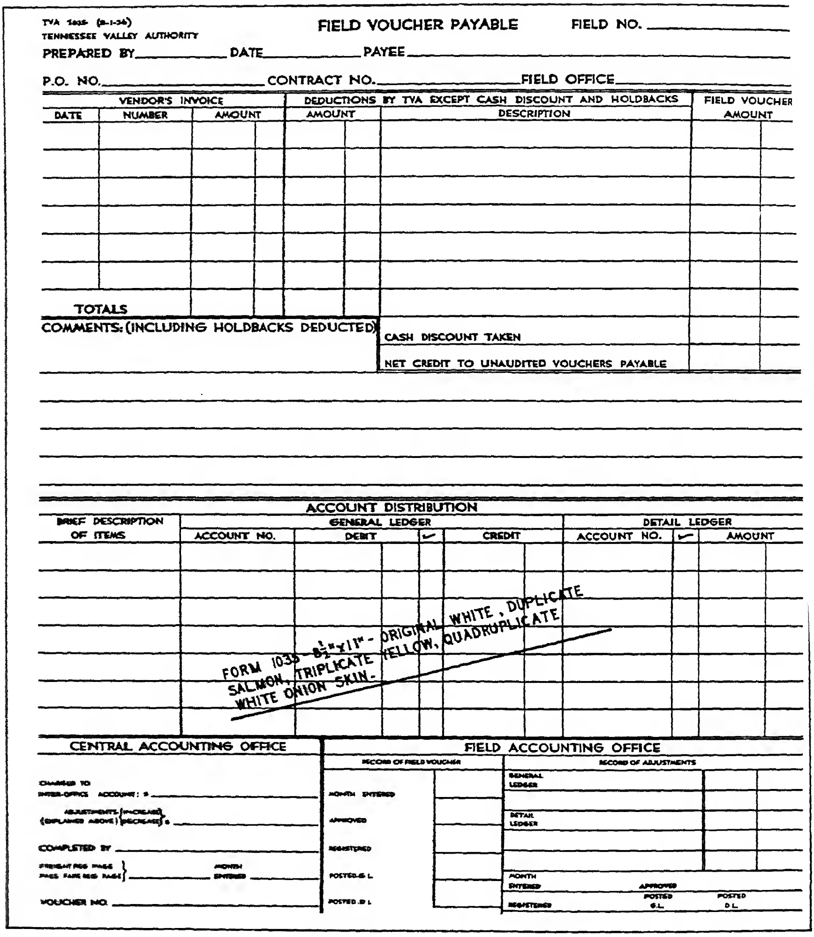


FIGURE 277.—*Public voucher for purchases and services other than personal.*

were kept separately. To secure the cost of such completely assembled plants, it was necessary to obtain the cost accumulated in both the plant and equipment accounts. Only the account for cableways (NDP-7) included both plant and equipment cost for the entire installation.



TVA 1035 (8-1-36)
TENNESSEE VALLEY AUTHORITY

FIELD VOUCHER PAYABLE — FIELD NO. ____

PREPARED BY ____ DATE ____ PAYEE ____

P.O. NO. ____ CONTRACT NO. ____ FIELD OFFICE ____

VENDOR'S INVOICE			DEDUCTIONS BY TVA EXCEPT CASH DISCOUNT AND HOLDBACKS		FIELD VOUCHER
DATE	NUMBER	AMOUNT	AMOUNT	DESCRIPTION	AMOUNT
TOTALS					
COMMENTS: (INCLUDING HOLDBACKS DEDUCTED)				CASH DISCOUNT TAKEN	
				NET CREDIT TO UNAUDITED VOUCHERS PAYABLE	

ACCOUNT DISTRIBUTION

BRIEF DESCRIPTION OF ITEMS	GENERAL LEDGER				DETAIL LEDGER		
	ACCOUNT NO.	DEBIT	✓	CREDIT	ACCOUNT NO.	✓	AMOUNT

FORM 1035 – $8\frac{1}{2}$"×11" – ORIGINAL WHITE, DUPLICATE SALMON, TRIPLICATE YELLOW, QUADRUPLICATE WHITE ONION SKIN.

CENTRAL ACCOUNTING OFFICE

CHARGED TO INTER-OFFICE ACCOUNT: $ ____
ADJUSTMENTS (EXPLAINED ABOVE) {INCREASE / DECREASE} $ ____
COMPLETED BY ____
FREIGHT REG. PAGE / PASS. FARE REG. PAGE ____ MONTH ENTERED ____
VOUCHER NO. ____

FIELD ACCOUNTING OFFICE

RECORD OF FIELD VOUCHER: MONTH ENTERED; APPROVED; REGISTERED; POSTED G. L.; POSTED D. L.

RECORD OF ADJUSTMENTS: GENERAL LEDGER; DETAIL LEDGER; MONTH ENTERED; APPROVED; REGISTERED; POSTED G. L.; POSTED D. L.

FIGURE 278.—*Field voucher payable.*

Journal vouchers.

Journal vouchers (fig. 279) were numbered in series designated by letters, with the series lettered as follows:

A. Salaries and wages.
B. Materials for direct purchases, storeroom tickets, shipping tickets, etc.
C. Clearing accounts, such as machine shop and garage.
D. Depreciation.
E. Miscellaneous items, such as adjustments and corrections.
F. Equipment and plant items transferred and sales.

The face of each journal voucher contained the entry to be made, which was supported by the attachment of work sheets and other data with sufficient reference for tracing the entry to its source. Posting to general ledger control, investments, and liability accounts was made from the face of the journals, and to detailed cost ledgers from the journal when itemized, but usually from recapitulation sheets attached to the voucher.

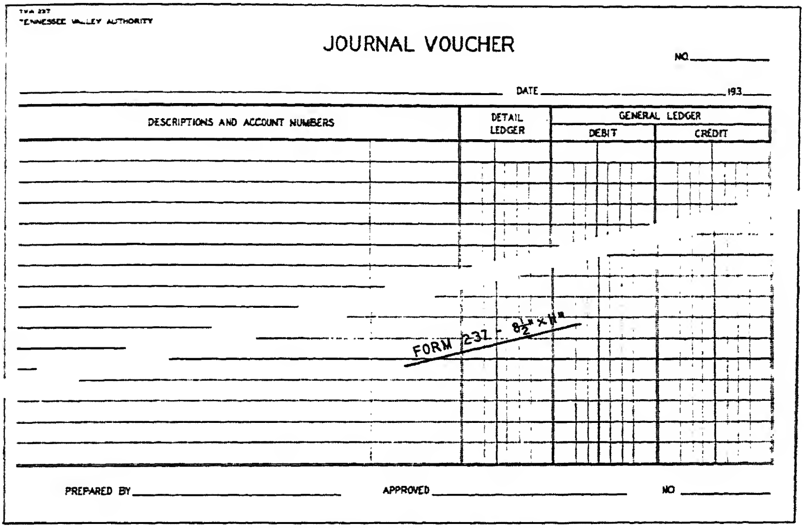


FIGURE 279.—*Journal voucher.*

Accounting for personal services.

The main objectives in accounting for personal services were (*a*) ascertainment of labor earnings for pay roll purposes, and (*b*) distribution to proper cost accounts. The method of accounting for annual and hourly employees was similar, except that separate pay rolls were maintained and methods of distribution differed.

Hourly employees.—The basis for the hourly employees' time distribution was the foremen's time cards (fig. 280). These cards, which were made out by the foremen, carried the badge number of each man in their group, giving a full description of the work being done and the number of hours worked. They were delivered to the time office by the foremen at the end of each shift. After the cost engineers assigned the account number, the time office rated and extended the amounts and balanced the hours and money daily with that shown on the posting sheet made up by the time checker. All overtime work was reported on foremen's time cards (form 149C—same as fig. 280 except for color) which required the approval of the superintendent. Metal time checks for all workers were kept at the time office on a specially prepared board. Employees were required

to call at the time office for their time check at the beginning of their shift and to return it at the end of the shift. Twice each shift a time checker made a tour of the job spotting the men at work as a check against the pay roll. The time checker's report (fig. 281) was prepared in duplicate from the check board after the workmen had checked in on their shift, and the copy was carried in the field by the checker. After he had completed his field check, he inserted the number of hours worked by each employee on the original, the time being posted by the pay-roll clerk from the original to the employee's ledger card (fig. 282).

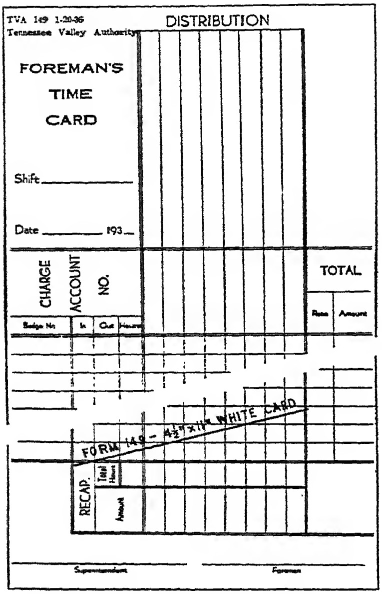

TVA 149 1-20-36
Tennessee Valley Authority
DISTRIBUTION
FOREMAN'S TIME CARD
Shift
Date 193
CHARGE ACCOUNT NO.
TOTAL
Rate Amount
Badge No. In Out Hours
FORM 149 - 4½"x11" WHITE CARD
RECAP. Total Hours Amount
Superintendent
Foreman

FIGURE 280.—*Foreman's time card.*

The responsibility of coding the foremen's time cards for distribution rested with the cost engineers as they were able to detect erroneous reports because of their intimate contact with the job. These were discussed with the foremen. When ambiguous or obscure descriptions were given, the cost engineers were able, in many instances, to clarify the description and to code the charges properly without delay.

A "labor ledger" for a number of feature accounts was kept by the cost engineers to facilitate analysis of labor charges made necessary by the demand for cost information in greater detail than was provided by the classification of accounts. In this "labor ledger" one sheet was reserved for each account on which the ledger was kept, and in it were entered daily the details for each labor charge including the name of the foreman, the detailed work description, and the amount in dollars for each item of work charged to the account. Since the "labor ledger" was kept for only part of the accounts, it was not incorporated in the accounting system. However, aside from its main purpose of facilitating analysis of accounts, it served the purpose of providing a check against obvious errors in coding.

Annual employees.—Where possible, pay-roll charges for foremen and general foremen were charged direct to features or to clearing accounts that were later distributed to the features. Other allocations of annual pay-roll charges were largely estimates prepared by the cost engineers on distribution schedules in memorandum form. For the majority of the salaried employees, this was sufficiently accurate because most of them were engineers, inspectors, warehousemen, and accountants.

Pay rolls.—Each employee was issued brass tool checks which were used for checking materials from the warehouse, such as tools, rain-

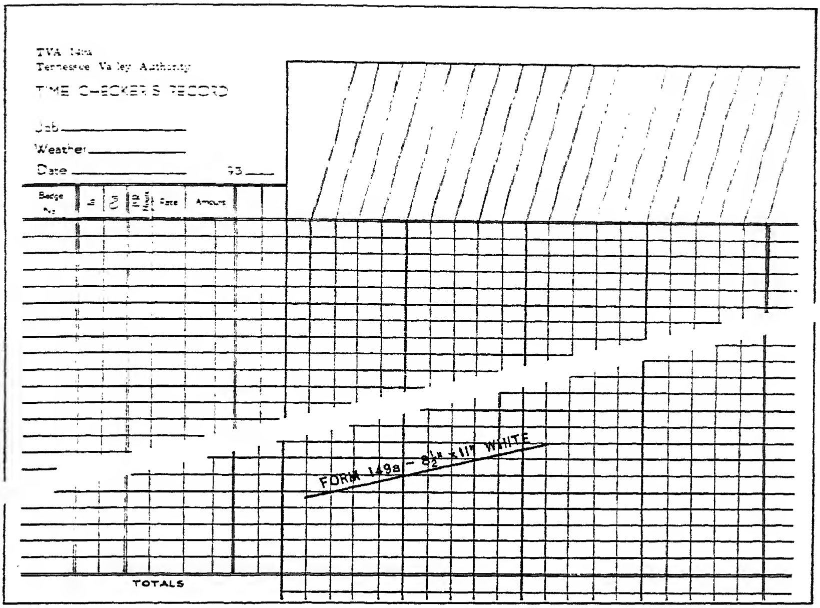


Figure 281.—*Time checker's record.*

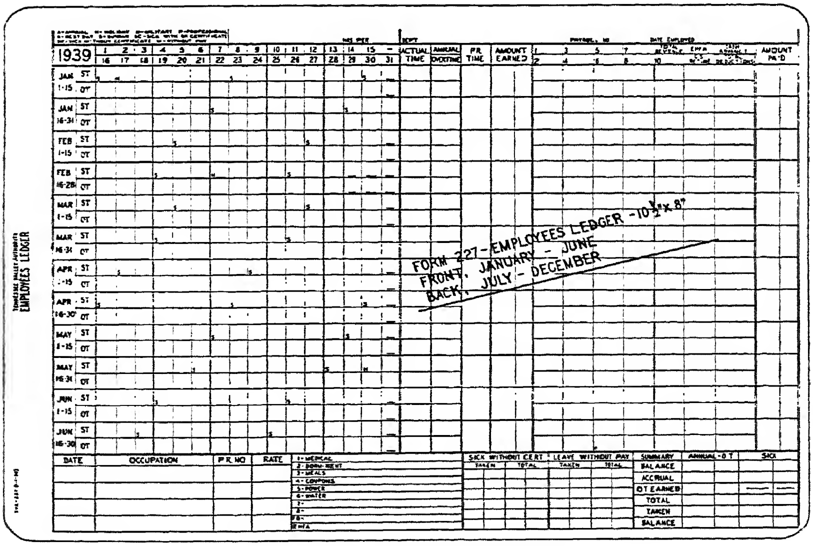


Figure 282.—*Employee's ledger.*

coats, and boots. When an employee lost these items, their value was deducted from his pay check.

Pay rolls were prepared twice monthly. The time was extended on the employee's ledger card; the deductions for camp services and other charges were made from the amount earned; and the net amount was extended in the "amount paid" column. The pay rolls, prepared on form 7 (fig. 283) were sent to Knoxville for auditing and the issuing of checks by the treasurer. Pay checks were delivered to the employees by the time office at the dam.

On termination or transfer to another project, employees' final checks were adjusted to include all deductions.

Accounting for material.

Materials accounting was logically divided in three steps: requisitioning and purchasing, receiving, and distributing and charging to accounts.

Requisitioning and purchasing.—All special and regular stock material was ordered by the chief storekeeper. For material not carried in regular stock, a list was sent to the chief storekeeper by the heads of the various sections, either engineering or construction, showing materials needed, quantity, time wanted, and approximate cost. Materials were bought on purchase requisition (fig. 284), which was prepared in four copies. The original and two copies were sent to the budget division, and one copy was retained for the warehouse files. From the purchase requisition, a standard purchase order was prepared (fig. 285). On the storeroom copy of the purchase requisition was noted the purchase order number, the account number to which the material was to be charged, and the vendor's name. When the material was later received, the checker's material receipt number was also noted on this copy of the requisition, together with the date received and how delivered. All information in regard to opening of bids and shipping was also noted on the requisition.

Receiving material.—The checker's material receipt (fig. 286) was used as a receiving report. It was prepared in four copies. A copy was retained for the warehouse files, the section ordering the material, and the chief material checker's files. This form showed the articles purchased, order or contract number, invoice price, handling charges (for stock materials) and account number when material was charged direct to the feature.

On the field copy of the purchase order (fig. 285) was entered the checker's receipt report number, when and how material was received, and the account number to which the material was to be charged. Then the inspector's copy of purchase order was executed by the receiving clerk and sent to the field accounting office, with the original copy of the checker's material receipt report and three copies of freight or express bill attached.

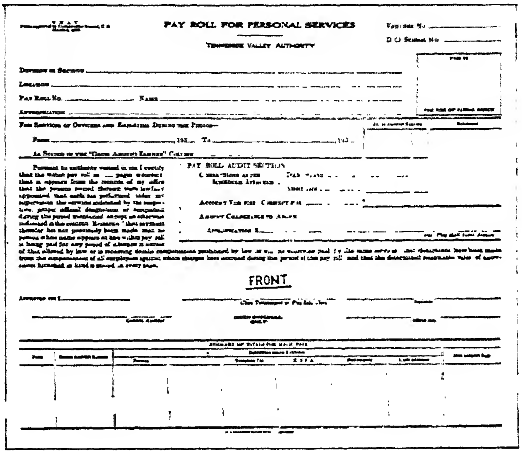
For partial shipments of materials, form 209 (fig. 287) was prepared in duplicate, one copy being sent to the accounting office with the original checker's material receipt and freight or express bills attached. The duplicate copy was retained by the storeroom office and attached to the field copy of the purchase order. On receiving

the final shipment, the same procedure was followed with the exception that final partial delivery report was attached to the inspector's copy of the purchase order.

Over, short, or defective material was reported on form 210 (fig. 288) which was prepared in four copies distributed to the traffic section, the inspection section, the central accounting office, and the warehouse files.

When material was received damaged by freight or express, proper notation was made on the railroad expense bill and information forwarded to the traffic section for filing of claim.

Materials for stock were entered on the stores ledger (fig. 289) direct from the copies of the invoice and shipping ticket. Special materials or direct charges were checked at the warehouse the same as stocked material but were not recorded on stock ledgers, being delivered and charged direct to the job for which they were used. (On later TVA jobs a separate warehouse was created for special materials ordered for the permanent structures. These were charged direct to their respective features upon arrival.)



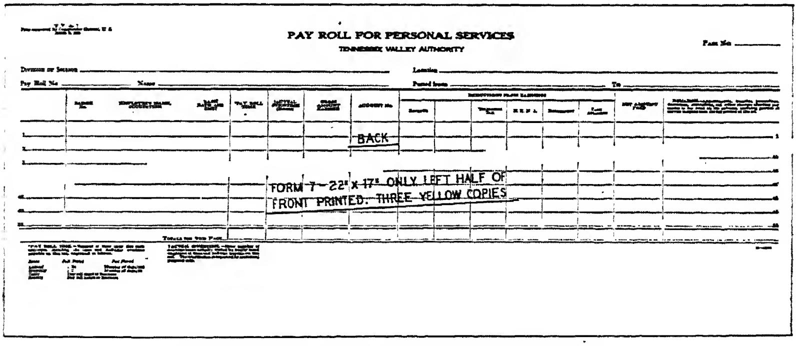
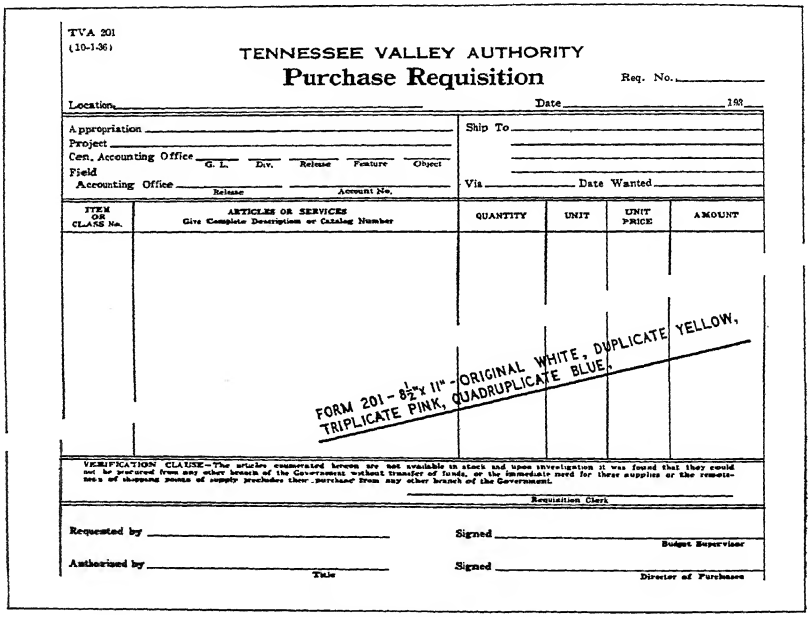


FIGURE 283.—*Pay roll for personal services.*

Direct charges.—The field voucher register was the basic document for the distribution of direct materials charges. With the checker's receiving report was prepared a field voucher payable, to which was

attached the invoice, checker's report, freight bills, and other pertinent data, as was previously explained, two copies of which were sent to the Knoxville office for payment. The coding for each field voucher was marked on its face. The voucher was listed in the field voucher register, showing the vendor's name, the total amount, and the account to which the charge was allocated. At the close of each month, the total amount chargeable to each account was posted in the cost ledger through the journal voucher.


TVA 201
(10-1-36)

TENNESSEE VALLEY AUTHORITY

Purchase Requisition

Req. No.

Location

Date 193

Appropriation
Project
Cen. Accounting Office G. L. Div. Release Feature Object
Field
Accounting Office Release Account No.

Ship To

Via Date Wanted

ITEM OR CLASS No.	ARTICLES OR SERVICES Give Complete Description or Catalog Number	QUANTITY	UNIT	UNIT PRICE	AMOUNT

FORM 201 - 8½" x 11" - ORIGINAL WHITE, DUPLICATE YELLOW, TRIPLICATE PINK, QUADRUPLICATE BLUE.

VERIFICATION CLAUSE—The articles enumerated hereon are not available in stock and upon investigation it was found that they could not be procured from any other branch of the Government without transfer of funds, or the immediate need for these supplies or the remoteness of shipping points of supply precludes their purchase from any other branch of the Government.

Requisition Clerk

Requested by

Signed
Budget Supervisor

Authorized by
Title

Signed
Director of Purchases

FIGURE 284.—*Purchase requisition.*

The responsibility of coding and classifying direct charges was given to the cost engineering organization as the members of this staff maintained an intimate contact with the job, a knowledge of the construction drawings, and enough technical background to know the relation of the different material items to the installation as a whole.

Warehouse issues.—Materials were issued from the warehouse on materials requisition (fig. 290) showing material, quantity wanted, purpose for which required, and signed by the section foreman or other such authorized person. At the warehouse this data was copied in duplicate on form 575, materials requisitioned and issued, and the cost placed thereon from the stock ledgers.

After being coded by the cost engineers the original of form 575 (fig. 291), with the support of form 211 (fig. 290), was sent to the field accounting office for entering to the proper account. The duplicate copy of form 575 was retained for warehouse reference.

Equipment and plant capital accounts.

Equipment was received from manufacturers or dealers by direct purchase and from other TVA projects by shipping ticket and transfer voucher (fig. 292).

As previously mentioned, a separate group of accounts was assigned for equipment charges and another similar group for plant charges. Equipment consisted of all tools and machinery not installed as a part of the feature or item of work and used in the con-

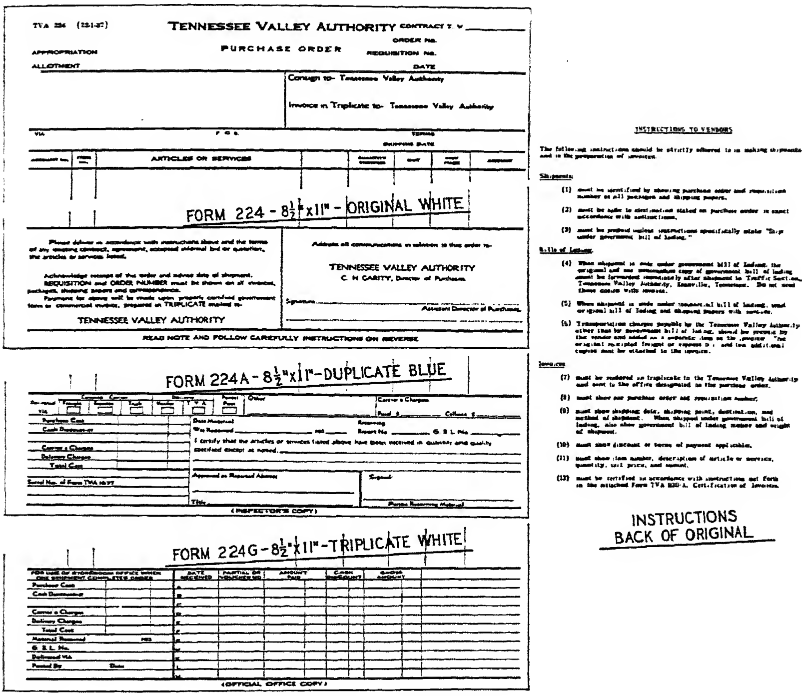


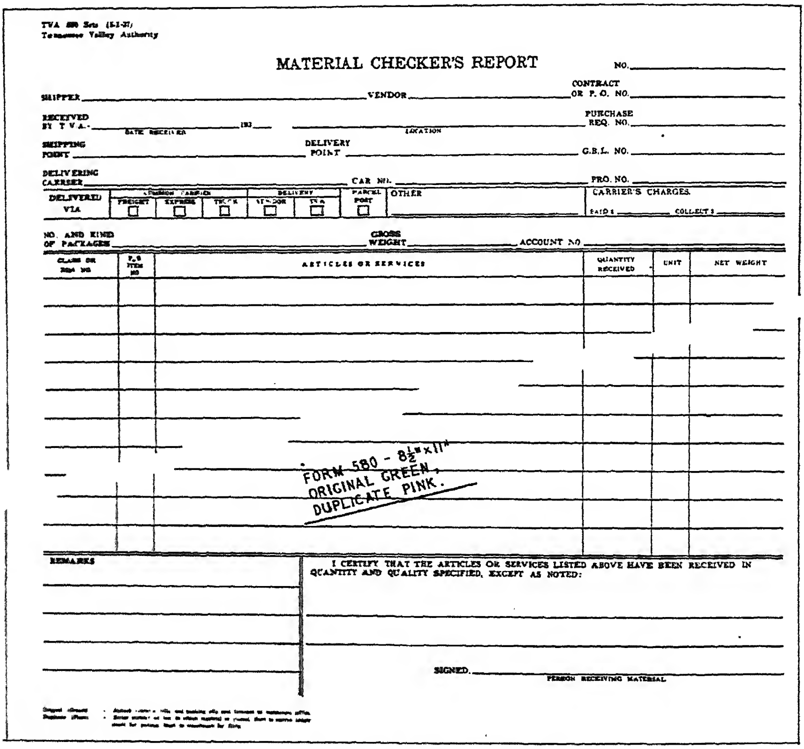
FIGURE 285.—*Purchase order.*

struction operations, such as steam shovels, tractors, trucks, and minor tools costing no more than $100 which retained their individual identity throughout their life, and whose useful life was estimated to exceed 6 months. Plant consisted of units of operation necessary and incidental to construction work such as mixer plants, crushing and screening plants, and cableways.

The depreciation policy formulated provided that the total cost of both plant and equipment should be absorbed in feature cost during the construction period, and schedules of equipment depreciation were developed from which a monthly charge was made to the work being performed and a credit given to an equipment and plant

reserve account. No direct credits were made to the group of accounts for equipment and plant charges, but the difference between the credit in the reserve and debit in the equipment and plant account represented the undepreciated amount invested in equipment and plants.

The policy of absorbing 100 percent of equipment depreciation into the job was followed throughout the job, the only exception being in the case of equipment acquired late in the the construction



Tennessee Valley Authority

MATERIAL CHECKER'S REPORT

NO. ____

SHIPPER ____ VENDOR ____ CONTRACT OR P. O. NO. ____

RECEIVED BY T V A. ____ 193__ ____ PURCHASE REQ. NO. ____

DATE RECEIVED — LOCATION

SHIPPING POINT ____ DELIVERY POINT ____ G.B.L. NO. ____

DELIVERING CARRIER ____ CAR NO. ____ PRO. NO. ____

DELIVERED VIA: COMMON CARRIER — FREIGHT ☐ EXPRESS ☐ TRUCK ☐; DELIVERY — VENDOR ☐ TVA ☐; PARCEL POST ☐; OTHER ____

CARRIER'S CHARGES: PAID $ ____ COLLECT $ ____

NO. AND KIND OF PACKAGES ____ GROSS WEIGHT ____ ACCOUNT NO ____

CLASS OR	F. S. ITEM NO	ARTICLES OR SERVICES	QUANTITY RECEIVED	UNIT	NET WEIGHT

FORM 580 - 8½"x11"
ORIGINAL GREEN,
DUPLICATE PINK.

REMARKS

I CERTIFY THAT THE ARTICLES OR SERVICES LISTED ABOVE HAVE BEEN RECEIVED IN QUANTITY AND QUALITY SPECIFIED, EXCEPT AS NOTED:

SIGNED ____
PERSON RECEIVING MATERIAL

FIGURE 286.—*Material checker's report.*

period for which it was definitely known there would be a need on other projects of the Authority.

Detailed engineering data and physical description of each piece of major equipment and each assembled plant were recorded by the warehouse on form 337 (fig. 293).

Equipment depreciation.—For all equipment, except that pertaining to an assembled plant, a date was established by which time it was anticipated that most of the heavy construction features would be completed, and by which time it would be advisable to have de-

preciated the equipment. Then the "straight line" method of depreciation was used, on a monthly basis, for each piece of equipment in order that the chosen date would be met. Equipment assigned to an assembled plant was depreciated on a production basis, at a uniform rate per unit of production.

The cost engineers prepared monthly a depreciation schedule, on which was listed by accounts the major pieces of equipment with lump sums for the balance, showing the month's depreciation and undepreciated balance. A special ledger (forms 239 and 239A, fig. 294) was kept for equipment, and this ledger was divided into two main groups—major equipment and minor equipment. The major

TVA 209 Sets (5-1-37)
Tennessee Valley Authority

PARTIAL DELIVERY REPORT

NO. ______

SHIPPER ______ VENDOR ______ CONTRACT OR P. O. NO. ______

RECEIVED BY T.V.A. ______ (DATE RECEIVED) 193__ ______ (LOCATION) PURCHASE REQ. NO ______

SHIPPING POINT ______ DELIVERY POINT ______ G.B.L. NO. ______

DELIVERING CARRIER ______ CAR NO. ______ PRO. NO. ______

DELIVERED VIA	COMMON CARRIER			DELIVERY		PARCEL POST	OTHER	CARRIER'S CHARGES
	FREIGHT ☐	EXPRESS ☐	TRUCK ☐	VENDOR ☐	TVA ☐	☐		PAID $ ______ COLLECT $ ______

NO. AND KIND OF PACKAGES ______ GROSS WEIGHT ______ ACCOUNT NO ______

CLASS OR BIN NO.	P. O. ITEM NO.	ARTICLES OR SERVICES	QUANTITY RECEIVED	UNIT	UNIT PRICE	AMOUNT

FORM 209 - 8½" x 11" ORIGINAL WHITE, DUPLICATE GREEN, TRIPLICATE SALMON, QUADRUPLICATE PINK.

COST OF MATERIAL RECEIVED:
PURCHASE COST $ ______
CASH DISCOUNT-CR. ______
______
CARRIER'S CHARGES ______
DELIVERY CHARGES ______
TOTAL COST ______
ENTERED IN STORES LEDGER ______ 193__ BY ______

I CERTIFY THAT THE ARTICLES OR SERVICES LISTED ABOVE HAVE BEEN RECEIVED IN QUANTITY AND QUALITY SPECIFIED, EXCEPT AS NOTED.

APPROVED AS REPORTED ABOVE: ______

SIGNED: ______ PERSON RECEIVING MATERIAL

Original (White)
Duplicate (Green) - - Attach copies of carrier's bill and packing slip and forward to warehouse office; white and salmon copies to accounting office.
Triplicate (Salmon)
Quadruplicate (Pink) - - Enter number of bin in which material is placed, then to stores ledger clerk for posting, then to warehouse for filing.

FIGURE 287.—*Partial delivery report.*

equipment group was subdivided in 24 classes, and the minor equipment group was subdivided in 20 classes. A detail of the subdivisions of these two groups follows. (See lists on page 581.)

A monthly statement was prepared from the equipment ledger for depreciation purposes and cleared through a series of journal vouchers. This statement listed each major item that was used on various parts of the job or assigned to some specific plant or shop. On this statement a column was provided for equipment account number, description, TVA number, cost to the project, depreciation to date, current depreciation, and charge account number.

TVA Form 210 (1-38)
TENNESSEE VALLEY AUTHORITY

NO ____________

OVER, SHORT, OR DEFECTIVE REPORT

(ALL CONTACTS WITH VENDORS WILL BE MADE BY MATERIALS DEPARTMENT)

DATE ____________ 19 ____

SHIPPER ____________ VENDOR ____________ CONTRACT OR P O NO ____________

RECEIVED BY T V A ____________ 19 ____ ____________ PURCHASE REQUISITION NO ____________
DATE RECEIVED RECEIVING POINT

SHIPPING POINT ____________ DELIVERY POINT ____________ PREPAID OR B B L NO ____________

DELIVERING CARRIER ____________ CAR NO AND INITIALS ____________ FREIGHT BILL NO ____________

WHEN CAR IS SEALED STATE SEAL NO AND INITIAL ____________ NAME OF PERSON BREAKING SEAL ____________

P O ITEM NO	LETTER CODE	DESCRIBE MATERIAL WITH DEFECT VARIANCE, ETC. APPLICABLE TO EACH ITEM	TOTAL QUANTITY RECEIVED	TOTAL QUANTITY ORDERED	DIFFERENCE OVER	DIFFERENCE SHORT	DIFFERENCE DEFECTIVE

FORM 210 - 8½"x11" - ORIGINAL WHITE, DUPLICATE BLUE, TRIPLICATE GREEN, QUADRUPLICATE PINK, QUINTUPLICATE YELLOW.

INDICATE BY FOLLOWING LETTER IN LETTER CODE COLUMN, WHETHER

A BALANCE OF ORDER SHOULD BE CANCELED
B ITEMS SHORT ON ORDER SHOULD BE OBTAINED
C MATERIALS RECEIVED IS ACCEPTABLE TO REQUISITIONER AS A SUBSTITUTE FOR THAT ORDERED
D MATERIAL RECEIVED MUST BE RETURNED TO VENDOR AND THE MATERIAL FURNISHED AS ORDERED
E-1 MATERIAL DAMAGED OR LOST IN TRANSIT -- DESCRIBE EACH ITEM SEPARATELY - REPLACEMENT WANTED
E-2 MATERIAL DAMAGED OR LOST IN TRANSIT -- DESCRIBE EACH ITEM SEPARATELY - DO NOT REPLACE
F-1 MATERIAL OTHERWISE DEFECTIVE -- DESCRIBE EACH ITEM SEPARATELY -- REPLACEMENT WANTED
F-2 MATERIAL OTHERWISE DEFECTIVE -- DESCRIBE EACH ITEM SEPARATELY -- DO NOT REPLACE

DISTRIBUTION OF COPIES:
ORIGINAL - ATTACH TO RECEIVING REPORT FOR CENTRAL ACCOUNTING DIVISION
DUPLICATE - TO FIELD OR LOCAL ACCOUNTING OFFICE
TRIPLICATE - } TO MATERIALS' DEPARTMENT OFFICE THAT ORIGINATED THE PURCHASE ORDER. { TO KNOXVILLE MATERIALS DEPT. OFFICE WITH FREIGHT BILL AND CLAIM PAPERS ATTACHED.
QUADRUPLICATE - } TO MATERIALS' DEPARTMENT OFFICE THAT ORIGINATED THE PURCHASE ORDER. { RETAINED BY ORIGINATING MATERIALS DEPARTMENT OFFICE
QUINTUPLICATE - RETAINED BY STOREROOM OR OFFICE ORIGINATING THIS FORM

SIGNED ____________
PERSON RECEIVING MATERIAL

APPROVED AS REPORTED ABOVE:

____
SIGNATURE

____
TITLE

(SEE INSTRUCTIONS ON REVERSE SIDE OF ORIGINAL)

INSTRUCTIONS

This form shall be used to report: (a) Overages or shortages in quantities of material ordered that are in excess of reasonable differences, (b) Material that does not meet the specifications stated in the purchase order, and (c) All defective or damaged material received. When space on the form is not sufficient to include a full report of the material in question, additional sheets should be used and attached to each copy of this form.

Damage or Shortage in Transit

Damage or shortage in transit must at once be called to the attention of agent for transportation company. If loss or damage is concealed, the agent must be offered an opportunity to inspect the packages. In all cases, he must be requested to make proper notation on all copies of freight or express bill. One copy of such bill should be attached to report. Claim papers composed of Forms TVA 387 or TVA 389 for shortage and/or TVA 387 or TVA 390 for damage, in quadruplicate, must be attached to report.

Defective, or Not to Specifications

Materials or equipment received should be inspected immediately upon receipt in an effort to detect imperfections and items that do not meet specifications. Whenever possible, this class of material should not be accepted from the delivering carrier, except small shipments which should be accepted but held in the original package in the storeroom until shipping instructions are received. The report on this material should be very specific, to enable the Procurement Section of the Materials Division to properly handle the matter with the vendor.

Disposition of Copies of this Report

Regardless of the nature of the report, all copies, with freight bills and claim reports, must be distributed as indicated on the reverse side hereof. The originating purchasing offices are indicated by purchase order number prefixes, as follows: C- Chattanooga; GD - Guntersville Dam; K - Knoxville; N - Wilson Dam; PD - Pickwick Dam; W - also to Knoxville. The Knoxville office originates all purchase contracts designated as TV.

INSTRUCTIONS BACK OF ORIGINAL

FIGURE 288.—*Over, short, or defective report.*

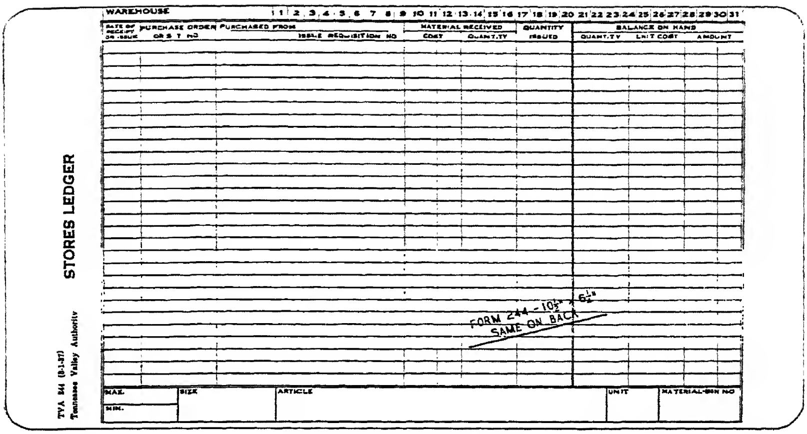


FIGURE 289.—*Stores ledger.*

Major Equipment Subdivisions

Air compressor.
Concrete equipment, portable mixers, etc.
Derricks and hoists.
Engineering instruments, transits, etc.
Excavating machinery, shovels.
Transfer cars, rail.
Locomotives and cranes.
Motors, large.
Trucks, tractors, and trailers.
Office equipment—Machines.
Office furniture and fixtures.
Pumps, large.
Shop Machinery—Lathes, drill, sharpener, arc welders, etc.
Transformers.
Wagon drills, core drills, and jackhammers.
Marine equipment—Boats.
Concrete vibrators.
Road rollers and scrapers.
Gasoline pumps and tanks.
Special equipment for removal of farm families.
Crusher plant equipment, crushers, motors, transformers, etc.
Mixer plant equipment—Mixers, motors, batchers, etc.
Sand plant equipment, hammer mills, motors, etc.
Cement pumps, etc., for handling bulk cement.

Minor Equipment Subdivisions

Blocks.
Personal equipment, boots, raincoats, etc.
Concreting tools, small.
Digging tools, picks, shovels, etc.
Engineering tools, small.
Electrical instruments, small.
Hose—Water, steam, or air, not part of plant.
Manila rope for general use.
Pipe and fittings, not included in water system.
Shop tools, small.
Warehouse equipment, trucks, scales, etc.
Wire rope, chain, etc.—Not chargeable to work or plant.
Woodworking tools.
Drilling tools, small.
Tarpaulins.
Reservoir clearing equipment.
Miscellaneous small equipment.
Laboratory testing equipment, large and small.
Carpenter shop equipment, large and small.
Relocation equipment, includes all heavy equipment such as trucks, tractors, shovels, pumps, compressors, etc.

TVA 211
Tennessee Valley Authority

STORES REQUISITION

No. ____________

DATE ______________________ 193___ DELIVER TO ______________________________

QUANTITY WANTED	ARTICLE
	FORM 211 – $5\frac{1}{2}'' \times 4\frac{1}{4}''$

WANTED FOR ACCT. NO.

FILLED ______________________ 193___

SIGNED ______________________________ BY ______________________________
SUPERINTENDENT
FOREMAN

FIGURE 290.—*Stores requisition.*

TENNESSEE VALLEY AUTHORITY

MATERIALS REQUISITIONED AND ISSUED No. M

WAREHOUSE ______________________________ DATE ______________________ 193___

DELIVER TO ______________________________ ACCOUNT NO. ______________________

LOCATION ______________________________ SHOP-JOB OR WORK ORDER NO. ______________________

WANTED FOR ______________________________

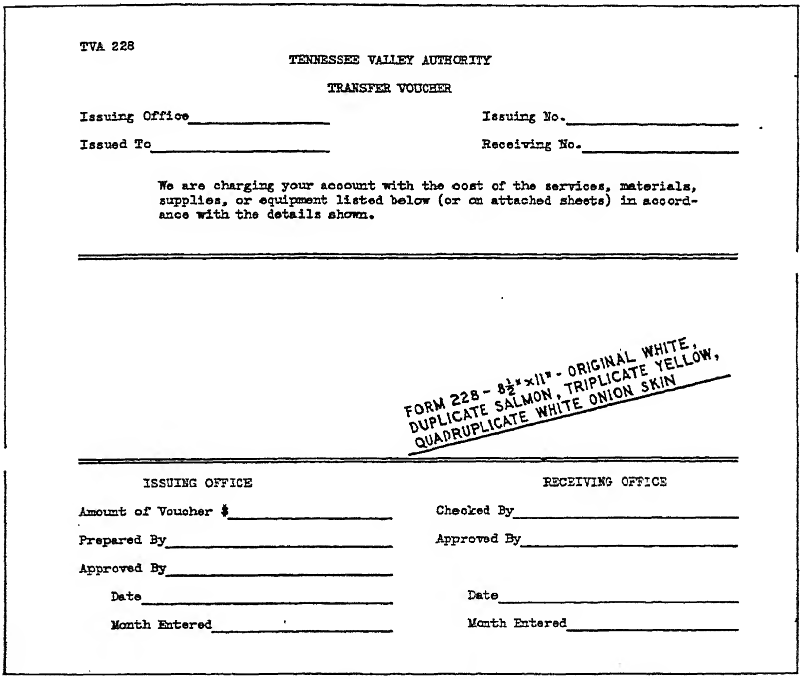
QUANTITY ORDERED	ARTICLE	MATERIAL OR BIN No.	QUANTITY DELIVERED	UNIT	UNIT PRICE	AMOUNT
	FORM 575 – $8'' \times 5''$ ORIGINAL WHITE DUPLICATE SALMON. TRIPLICATE YELLOW.					
					TOTAL AMOUNT	
					% STORES EXPENSE	
					TOTAL CHARGE	

APPROVED ______________________ DIVISION ______________________

RECEIVED ______________________ DATE ______________ FILLED BY ______________________

FIGURE 291.—*Materials requisitioned and issued.*

Plant depreciation.—Estimates were made by the cost engineers at the beginning of operations of the total quantities to be involved with each plant. Then monthly amounts were depreciated into plant operating accounts which had the same relation to total plant cost as did the monthly quantities to the estimated total job quantities. Plant charges for power, water, and air systems, and for plant buildings, were depreciated at a uniform monthly rate so as to absorb their costs by the end of the heavy construction operations. Plant roads and bridges were not depreciated and cleared to the feature accounts



TVA 228

TENNESSEE VALLEY AUTHORITY

TRANSFER VOUCHER

Issuing Office__________ Issuing No.__________

Issued To__________ Receiving No.__________

We are charging your account with the cost of the services, materials, supplies, or equipment listed below (or on attached sheets) in accordance with the details shown.

FORM 228 - 8½"x11" - ORIGINAL WHITE, DUPLICATE SALMON, TRIPLICATE YELLOW, QUADRUPLICATE WHITE ONION SKIN

ISSUING OFFICE	RECEIVING OFFICE
Amount of Voucher $______	Checked By______
Prepared By______	Approved By______
Approved By______	
Date______	Date______
Month Entered______	Month Entered______

FIGURE 292.—*Transfer voucher.*

but were written off completely at the close of construction, their cost being distributed over the entire job on the basis of total feature costs.

Equipment and plant disposals.—As the work drew to a close, much of the equipment and plant was still useful and was transferred to other projects. Available equipment was reported on form 1,337 (fig. 295), and transfers were effected through the cooperation of the construction plant organization in Knoxville, values being determined after an appraisal by this organization with the approval of both the receiving and issuing projects. Credits for equipment transferred were placed on the project books to special accounts in the equipment group designated as "equipment transferred or sold," and

"plant transferred or sold," this credit to be applied to reduce the final project investment in plant and equipment.

At the time of closing the construction books, schedules were prepared for spreading these credit accounts. The schedule for equipment listed by each one of the equipment accounts, the principal pieces of construction equipment showing its description, original cost,

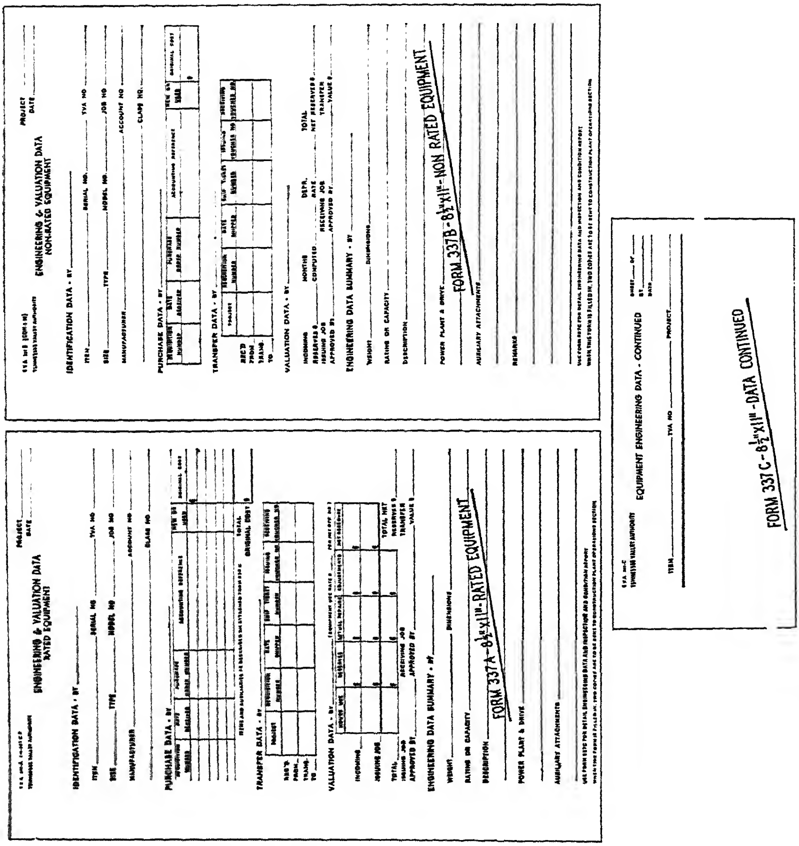


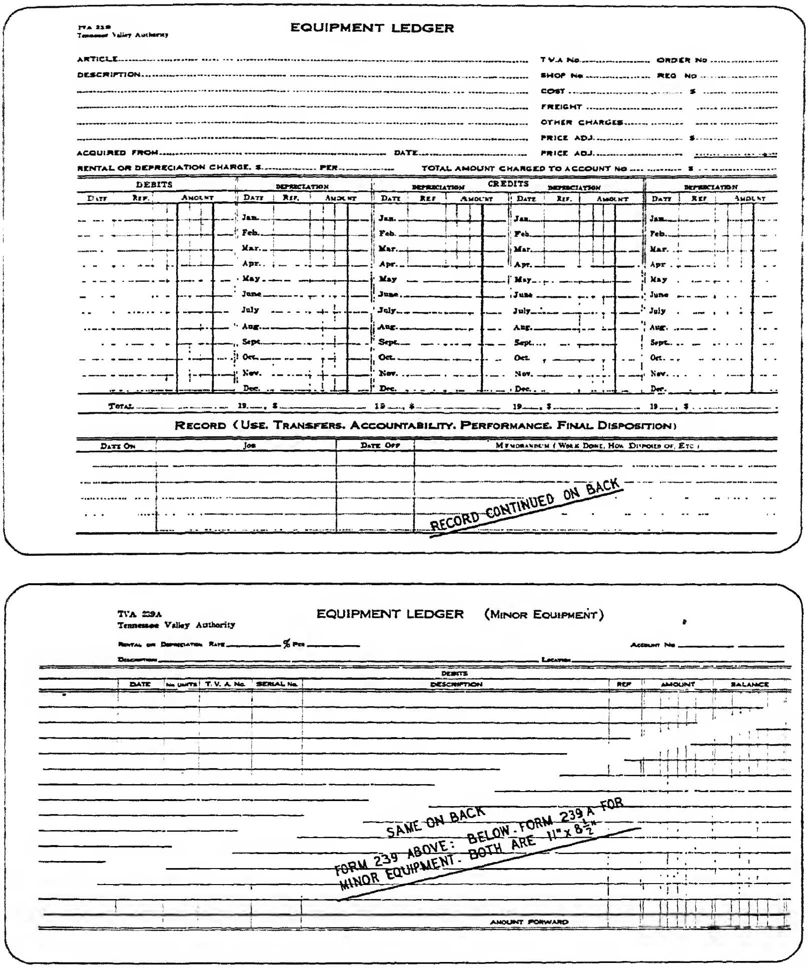
FIGURE 293.—*Engineering and valuation data, rated and nonrated equipment.*

amount depreciated, and transfer value. An adjustment was then made to both the feature or clearing account to which depreciation had been allocated and to the corresponding depreciation reserve account. For small and minor equipment for which it was not practical to allocate the adjustment to features or clearing accounts, the credit was accumulated in a work order to be distributed to all features on the basis of total cost.

A similar adjustment for major plant accounts was made to the plant operating accounts. The credit for small, miscellaneous, un-

identifiable items was applied to the work order mentioned in the preceding paragraph to be distributed to the feature accounts.

For equipment transferred to other projects, the storeroom prepared a shipping ticket (fig. 296). Upon receiving a copy of this



TVA 239
Tennessee Valley Authority

EQUIPMENT LEDGER

ARTICLE ... T.V.A No. ... ORDER No ...
DESCRIPTION ... SHOP No ... REQ No ...
COST ... $ ...
FREIGHT ...
OTHER CHARGES ...
PRICE ADJ. ... $ ...
ACQUIRED FROM ... DATE ... PRICE ADJ. ...
RENTAL OR DEPRECIATION CHARGE. $ ... PER ... TOTAL AMOUNT CHARGED TO ACCOUNT No ... $ ...

DEBITS			DEPRECIATION			CREDITS DEPRECIATION			DEPRECIATION			DEPRECIATION		
Date	Ref.	Amount	Date	Ref.	Amount	Date	Ref	Amount	Date	Ref.	Amount	Date	Ref	Amount
			Jan.			Jan.			Jan.			Jan.		
			Feb.			Feb.			Feb.			Feb.		
			Mar.			Mar.			Mar.			Mar.		
			Apr.			Apr.			Apr.			Apr.		
			May			May			May			May		
			June			June			June			June		
			July			July			July			July		
			Aug.			Aug.			Aug.			Aug.		
			Sept.			Sept.			Sept.			Sept.		
			Oct.			Oct.			Oct.			Oct.		
			Nov.			Nov.			Nov.			Nov.		
			Dec.			Dec.			Dec.			Dec.		
Total			19 , $			19 , $			19 , $			19 , $		

RECORD (USE. TRANSFERS. ACCOUNTABILITY. PERFORMANCE. FINAL DISPOSITION)

Date On	Job	Date Off	Memorandum (Work Done, How Disposed of, Etc.)

RECORD CONTINUED ON BACK

TVA 239A
Tennessee Valley Authority

EQUIPMENT LEDGER (MINOR EQUIPMENT)

Rental or Depreciation Rate ... % Per ... Account No ...
Description ... Location ...

DATE	No. UNITS	T. V. A. No.	SERIAL No.	DEBITS DESCRIPTION	REF	AMOUNT	BALANCE
				AMOUNT FORWARD			

SAME ON BACK
FORM 239 ABOVE: BELOW FORM 239 A FOR MINOR EQUIPMENT. BOTH ARE 11" x 8½"

Figure 294.—*Equipment ledger.*

shipping ticket, the accounting office prepared the shipper's report (fig. 297), two copies of which were retained by the accounting office, one for attaching to the journal voucher transferring the charge, and the other to be placed in a shipper's report file.

The original copy of this report was used for the basis of establishing transfer values, and an equipment transfer memorandum (fig. 298) was prepared showing the price the receiving job was expected to pay. After the issuing and receiving jobs had approved this established price, copies of the equipment transfer memorandum were sent to the issuing accounting office for authorization to charge the receiving job for the amount agreed upon. Equipment and plant transfer values for large assembled plants such as the rock crushing and screening plant and the cableway, were generally established by executive decision.

TVA 1337
Tennessee Valley Authority

CONSTRUCTION PLANT DIVISION

AVAILABLE EQUIPMENT REPORT

Project__________ Date__________193___

TVA NO.	DESCRIPTION	CONDITION*	AVAILABLE DATE

FORM 1337 - 8½" x 11"

*A - (New) - In service not more than three months.
B - (Good) - Major repairs made, worn parts replaced.
C - (Fair) - Running condition.
D - (Poor) - Needs repairs to put in running condition.
E - (Junk) - Reconditioning not justified for job use.

____
Submitted

____
Approved

FIGURE 295.—*Available equipment report.*

Equipment valuation.—Considerable study was given to the transfer of the construction plant equipment. Since the plant contained varied equipment, it was necessary to determine suitable units of service on which to distribute ownership investment. For this purpose the equipment was classified in two groups. Any equipment adapted to recording of hours, service, and repairs was considered in one group, and items not well adapted to such records in a second group. This classification was briefly as follows:

	Service unit	
	Hourly or major	Monthly or minor
Hauling plant	Locomotives, tractors, trucks	Cars, trailers, wagons.
Drilling and pneumatic plant.	Core drills	Compressors, drills, air receivers.
Grading plant	Ditchers, graders, rollers, shovels	Buckets, scarifiers, scrapers, tampers.
Hoisting plant	Cranes	Derricks, hoists, cableways.
Unwatering plant		Pumps.
Marine plant	Towboats, derrick boats	Barges, launches, outboard motors.
Aggregate plant		Bins, crushers, hoppers.
Cement plant		Bins, hoppers, pumps.
Concrete plant		Bins, batchers, mixers, vibrators.
Miscellaneous plant	Portable loaders and conveyors	Boilers, light plants, shop equipment.

TVA 144
Tennessee Valley Authority

SHIPPING TICKET No. A

Date ____

Shipper ____ Point of Origin ____

From Warehouse ____ Authority ____

Ship to

Date Shipped ____
B\L No. ____ Date ____
Made by ____
Car No. ____ Car Seals ____
Routing ____

Bill to FORM 144 - 8½" x 11" - ORIGINAL WHITE, DUPLICATE BLUE, TRIPLICATE GREEN, QUADRUPLICATE SALMON, QUINTUPLICATE YELLOW, SEXTUPLICATE PINK.

Charge ____ Account No. ____ Amount ____ Credit ____ Account No. ____ Amount ____

BIN or SYMBOL	DESCRIPTION (Give serial numbers, numbers of orders on which containers were originally received, number of packages, weights, etc.)	MATERIAL NUMBER Class & Item	QUANTITY DELIVERED	UNIT	UNIT PRICE	AMOUNT

ORIGINAL
To Accounting Office to record the transfer

DUPLICATE
To Accounting Office to be attached to transfer voucher

TRIPLICATE
To Property Section, Knoxville

Record cards posted ____

QUADRUPLICATE - SHIPPING NOTICE
Enclose this copy with material shipped as a packing slip whenever possible, or forward direct to consignee with inspector's copy.

THIS SPACE FOR USE OF RECEIVING OFFICE:
COST ____
CARRIER'S CHARGES ____
DELIVERY CHARGES ____
TOTAL COST ____

MATERIAL RECEIVED ____ 193 ____
VIA ____
CHECKED BY ____
POSTED TO STORES LEDGER ____

	By	Date
Selected		
Checked		
Packed		
Shipped		

QUINTUPLICATE
FILE COPY
Retained by originator

Stock cards posted ____

SEXTUPLICATE - INSPECTOR'S COPY
Forward this copy to consignee under separate cover.

The consignee shall complete this section of the form and forward to the accounting office keeping their accounting records.

MATERIAL RECEIVED ____ 193 ____
VIA ____

I certify that the articles or services listed above have been received in quantity and quality specified. Except as noted: ____
SIGNED: ____
TITLE: ____

FIGURE 296.—*Shipping ticket.*

All items are not included in this tabulation. However, it indicates the types of equipment included in each group of the classification. Plant assemblies representing major assets could generally be broken down into units classified as to type of equipment for depreciation of valuation purposes.

T.V.A. 324
TENNESSEE VALLEY AUTHORITY

EQUIPMENT SERVICE AND CONDITION
PROJECT: ____________

SHIPPER'S REPORT

T.V.A. EQUIPMENT NO: ________ ITEM: ____________

SHIPPED TO ____________ DATE: ________ SHIPPING TICKET NO: ________

RECEIVED FROM ____________ USED NEW DATE: ________

ORIGINAL COST TO PROJECT: ________ DESCRIPTION OF ADDITIONS

ADDITIONS MADE ON PROJECT: ________ ____________

ADDITIONS MADE ON PROJECT: ________ ____________

TOTAL COST TO PROJECT: ________ DEPRECIATION: ________ BOOK VALUE: ________

SERVICE REPORT

DATE PUT IN SERVICE ________ DATE SERVICE COMPLETED: ________

TYPE OF SERVICE ____________

____

WORK DONE (QUANTITIES) ____________

CONDITION REPORT

MAJOR REPAIRS AND REPLACEMENTS	DATE	APPROX. COST

FORM 1324 - 8½"x11"

DESCRIPTION FINAL OVERHAUL TOTAL COST FINAL OVERHAUL: ________

____

____

____

REMARKS: ____________

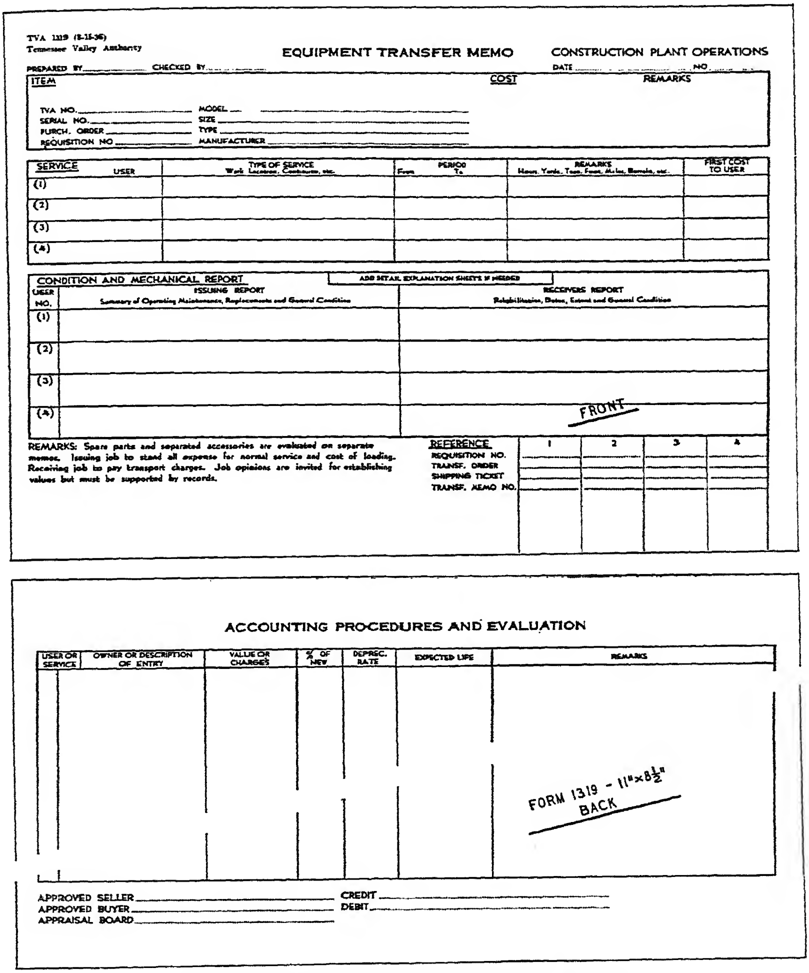
____

DATE: ________ SIGNED: ________

FIGURE 297.—*Shipper's report.*

Furthermore, it was necessary to determine the expected life of individual types of equipment in both classes. This problem involved two types of expected life, namely economical and TVA life. Economical life was determined on many units, mostly in the monthly or minor class by examination of service and repair records and con-

dition at time of transfer. On equipment whose utility was not completely exhausted on the Norris project, these records fairly well established the economic service life for the next user. On the other hand, some units mostly of a major nature were found to have an


TVA 1319 (8-15-36)
Tennessee Valley Authority

EQUIPMENT TRANSFER MEMO

CONSTRUCTION PLANT OPERATIONS

PREPARED BY ________ CHECKED BY ________ DATE ________ NO. ________

ITEM — COST — REMARKS

TVA NO. ________ MODEL ________
SERIAL NO. ________ SIZE ________
PURCH. ORDER ________ TYPE ________
REQUISITION NO ________ MANUFACTURER ________

SERVICE USER	TYPE OF SERVICE Work Location, Continuous, etc.	PERIOD From To	REMARKS Hours, Yards, Tons, Feet, Miles, Barrels, etc.	FIRST COST TO USER
(1)				
(2)				
(3)				
(4)				

CONDITION AND MECHANICAL REPORT — ADD DETAIL EXPLANATION SHEETS IF NEEDED

USER NO.	ISSUING REPORT Summary of Operating Maintenance, Replacements and General Condition	RECEIVERS REPORT Rehabilitation, Dates, Extent and General Condition
(1)		
(2)		
(3)		
(4)		FRONT

REMARKS: Spare parts and separated accessories are evaluated on separate memos. Issuing job to stand all expense for normal service and cost of loading. Receiving job to pay transport charges. Job opinions are invited for establishing values but must be supported by records.

REFERENCE	1	2	3	4
REQUISITION NO.				
TRANSF. ORDER				
SHIPPING TICKET				
TRANSF. MEMO NO.				

ACCOUNTING PROCEDURES AND EVALUATION

USER OR SERVICE	OWNER OR DESCRIPTION OF ENTRY	VALUE OR CHARGES	% OF NEW	DEPREC. RATE	EXPECTED LIFE	REMARKS
						FORM 1319 - 11"×8½" BACK

APPROVED SELLER ________ CREDIT ________
APPROVED BUYER ________ DEBIT ________
APPRAISAL BOARD ________

FIGURE 298.—*Equipment transfer memorandum.*

economic life exceeding the period of the Authority's scheduled construction in which case TVA life expectancy studies were made. On equipment of this nature the expected life was established as less than the economic life in order that the ownership investment might

be liquidated at the close of the scheduled service without unfavorably altering cost records.

In general, the class of minor equipment was found not to experience exorbitant expense of conditioning, and the service unit of calendar months was established for depreciation and valuation purposes. Straight line monthly depreciation rates based on life expectancy studies were used for this class of equipment, which represented the greatest number of items and involved relatively small ownership investments. By thus simplifying the treatment of the majority of equipment items, the major items which represented larger ownership investments could be given more detailed study.

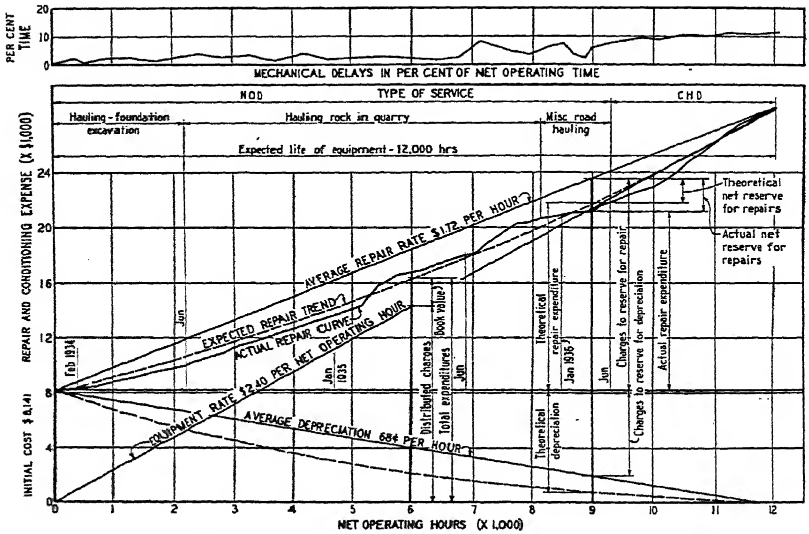


Figure 299.—*Economical rate depreciation.*

The most suitable unit of service for valuation of major units of equipment was found to be "net operating hours"; i. e., gross hours or hours that an operator is assigned to a machine for presumed active service less all delay time. Delay time was classified as mechanical, servicing, miscellaneous, and other delays and was usually recorded to the nearest quarter hour. Conditioning investment on major equipment was found to increase with an upward curved trend as additional units of service were recorded. As illustrated in figure 299, the lowest ownership rate was established by the curve when ownership investment (first cost plus repairs) was plotted as ordinates and units of service as abscissa. This experience on groups of similar items helped determine the economic life of the various units and from the average trend the depreciation curve used for valuation was obtained. Generally one of the following depreciation curves was found to be applicable to major equipment items:

Depreciation schedules

	Class A	Class B	Class C
	Percent	*Percent*	*Percent*
First quarter of expected life	44	40	34
Second quarter of expected life	31	32	30
Third quarter of expected life	19	19	21
Fourth quarter of expected life	6	9	15
Total	100	100	100

By applying these depreciation rates, the ownership investment was distributed fairly accurately in units of service to all using projects since each project was expected to absorb conditioning costs during the period of use on the project. Due allowance was always made for exceptionally good or bad condition of the equipment at the time of transfer.

From the experience at Norris it became evident that the original policy of 100 percent job write-off of construction plant equipment involved considerable extra accounting work in handling credits and was not the proper intent or logical method of handling the Authority's construction plant assets. The experience obtained in recording hours, service, and repairs, supplemented with experience from other Authority projects under construction at the same time, led to the devising of an automatic system of accounting and valuation of construction plant equipment for use on future projects of the Authority. In this system of investment control, equipment was classified into rated and nonrated items corresponding to major and minor units used to facilitate valuation of the Norris construction plant.

Equipment use rates were set up to cover cost of owning and conditioning rated or major equipment, and monthly depreciation percentage rates were set up to retire investments in nonrated or minor items. This system enabled construction plant investment accounts to show book values in line with intrinsic values of the equipment and automatically provided sound transfer values. It also enabled equitable distribution of ownership investment as work progressed, and each job was able to apply a uniform method of clearing equipment repair and depreciation expense.

Figure 300 shows the method adopted in determining use rates and transfer values on rated equipment. This system has also facilitated the scientific study of disposition of equipment, especially in regard to the economies of salvage or replacement at any time during the life of the equipment.

Equipment and plant operations.

As modern construction work is done largely by machinery, a considerable part of the construction cost is represented in the machinery operating accounts. The cost of such units of operation was kept in a group of accounts designated "clearing accounts," an account being maintained for each unit or group of similar units. The use of clearing accounts was also extended to other operations such as operation of electrical systems, water systems, and maintenance of general construction facilities. At the close of each period the charges as shown

on these clearing accounts were distributed, usually on the basis of use, but in some instances on the basis of labor or feature cost. The debits for clearing accounts were accumulated and totalled, and a separate account for credits was assigned so that the cost ledgers would always show the accumulative total cost as well as accumulative credits to any one operation. With such a system of accounting, there was inevitably a balance, but upon the completion of the work these balances were distributed to the feature accounts.

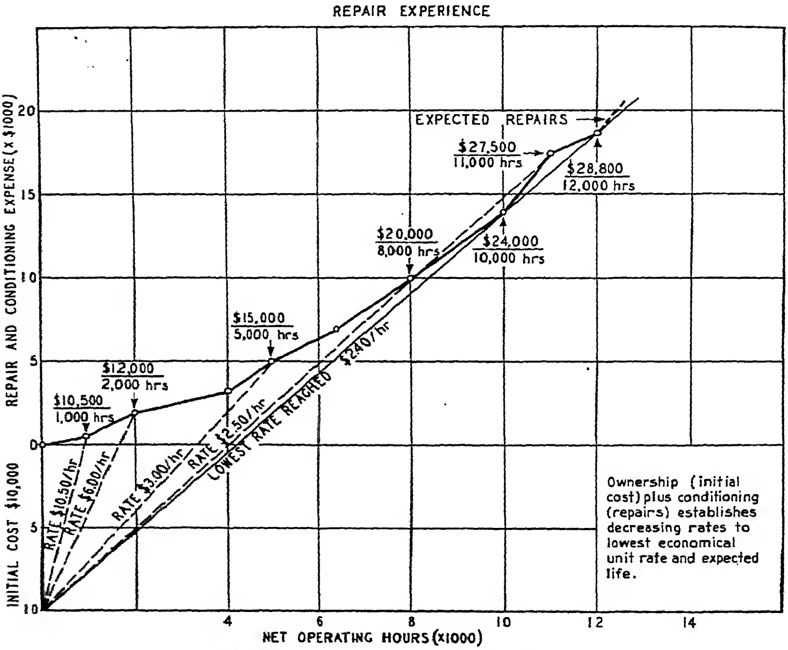


FIGURE 300.—*Equipment investment studies.*

In entering charges for plant and equipment use on cost ledgers, separate clearances and entries were made for charges from the quarry, coarse aggregate plant, fine aggregate plant, steam and handling systems, mixing plant, concrete hauling system, cableways, power systems, water systems, compressed air systems, job cleanup, and camp and ground. Charges for the use of the remainder of the construction equipment such as trucks, tractors, power shovels, and cranes were accumulated and posted in one entry for each period.

Hourly rates for distribution.—Rates for power shovels were obtained directly from the cost accountant while the rates for other plant and equipment were prepared by the cost engineers using the mobile expense report (fig. 301) as the basis for mobile equipment. Hourly rates for the cableway varied from month to month, being the product of monthly charges divided by total hours operated.

TVA 689
TENNESSEE VALLEY AUTHORITY

MOBILE EXPENSE

MACHINE .... T.V.A. NO. ......

INITIAL COST ....

DATE PURCHASED ....

PURCHASED FROM ....

PROJECT ....

NOTE:
"EARNINGS" IS "WORKING HOURS" MULTIPLIED BY RENTAL RATE.
"DEPRECIATION" IS "EARNINGS" MINUS "TOTAL OPERATING AND REPAIR COSTS."
"UNIT COST PER WORKING HOUR" IS "OPERATING" AND "REPAIR" COST DIVIDED BY "WORKING HOURS."

DATE .... RENTAL RATE .... PER ....
DATE .... RENTAL RATE .... PER ....
DATE .... RENTAL RATE .... PER ....

DATE	OPERATING COST						REPAIRS AND REPLACEMENTS					TOTAL OPERATING AND REPAIR COSTS	DEPRECIATION	EARNINGS	WORKING HOURS	TONS, CU.YDS. MILES, ETC.	UNIT COST PER		REMARKS	
	OPERATORS DRIVERS HELPERS	POWER OR FUEL	CYL. OIL	GREASE-ETC	TIRES-TUBES	MISC.	LABOR		PARTS		FOREMAN AND OVERHEAD						WORKING HOUR		GAL GAS	QUARTS OIL
							NORMAL	ACCIDENTAL	NORMAL	ACCIDENTAL										
BROUGHT FORWARD																				
TOTAL TO																				
TOTAL TO																				
TOTAL TO																				
TOTAL TO																				
TOTAL TO																				
TOTAL TO																				
TOTAL TO																				
TOTAL TO																				

FORM 689-16" x 11"

FIGURE 301.—*Mobile expense.*

Hourly rates for other equipment, however, were predetermined, and it was necessary to adjust them only a few times during the construction period to agree with actual costs.

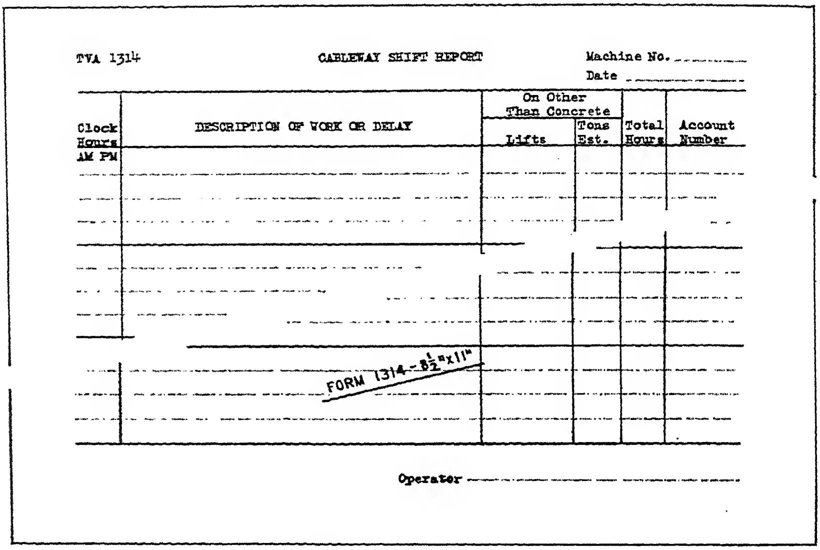


FIGURE 302.—*Cableway shift report.*

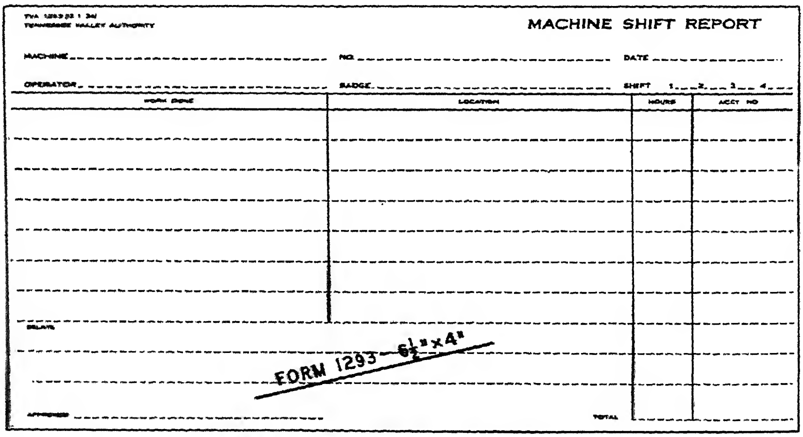


FIGURE 303.—*Machine shift report.*

Machine time reports.—Daily shift reports were made by the machine operators indicating their estimate of the time spent on each operation. With these as a basis it was possible to arrive at an

equitable distribution of each machine's operating expense. The cableway shift report (fig. 302) was designed especially to aid the operator in allocating charges correctly. Operating time for shovels

TVA 324A
Tennessee Valley Authority

EXCAVATION FOREMAN'S REPORT

FOREMAN ______ DATE ______

SHIFT ______

LOCATION ______

SHOVEL #1			SHOVEL #2			SHOVEL #3			SHOVEL #4			SHOVEL #6		
Hours worked	Loads	Yds. per load	Hours worked	Loads	Yds. per load	Hours worked	Loads	Yds. per load	Hours worked	Loads	Yds. per load	Hours worked	Loads	Yds. per load

DELAYS — CAUSE OF DELAY

SHOVEL #1. Hrs. ____ Min. ____ ______
" 2. " ____ " ____ ______
" 3. " ____ " ____ ______
" 4. " ____ " ____ ______
" 6. " ____ " ____ ______

TRUCKS		TRACTORS	
Job. No.	Hrs. Used	Job No.	Hrs. Used

MATERIAL HANDLED

SHOVEL 1 ______
" 2 ______
" 3 ______
" 4 ______
" 6 ______

FORM 324A - 8½" x 11"

FIGURE 304.—*Excavation foreman's report.*

was reported on the excavation foreman's report (fig. 304). Records of truck and tractor operations were reported on shift truck report (fig. 305).

Shop order systems.

Shop order systems were maintained for the machine shop, the garage shop, and the carpenter shop. However, the charges from the electric shop for labor and material were made direct to the

features on which the work was done because of the relative simplicity of operations and the smaller jobs performed. The general method of handling shop operating charges was to allocate to the

TVA 338 SHIFT TRUCK REPORT
Tennessee Valley Authority Project__________

Vehicle__________ No.__________ Date__________ 193__

License No.__________ Shift______ M. to______ M.

Mileage: Speedometer Reading--IN __________	Gasoline Issued ________ Gals.
Speedometer Reading-OUT __________	Oil Issued ________ Qts.
Miles Run	

Time Distribution

Trips or Loads - Description	Hours
Delays - (cause, length of time, etc.)	

Remarks (Attention needed on, etc.)	Total Hours
	Operator

FORM 338 – 8½"x11"

FIGURE 305.—*Shift truck report.*

proper shop operating account (set up in the clearing account group) all charges for direct or supervisory labor, material, and shop expense. The shop accounts were cleared by means of shop orders.

All requests for shop work were prepared in writing on stores requisition (fig. 290). Major and blanket shop orders were the two general types used.

Major shop orders.—Separate shop orders (fig. 306) were prepared for each major job. On these orders were posted quantities and costs of material used, time required, and labor rates. The time required

TVA 231
Tennessee Valley Authority SHOP ORDER
No.

TO FOREMAN OF DIVISION

You are requested to perform the following described work:

Deliver to Signed

Date Wanted Account Title

LABOR							MATERIAL				
DATE	EMPLOYEES NUMBERS					OPER. NO.	DATE	ITEMS	QUAN.	PRICE	AMOUNT
	HRS.	HRS.	HRS.	HRS.	HRS.						

FORM 231 - 8½" x 11" - ORIGINAL WHITE, DUPLICATE BLUE, TRIPLICATE WHITE CARD

TOTAL

SUMMARY	
LABOR	
MATERIAL	
COST	
OVERHEAD %	
TOTAL CHARGE	

DR.

WORK BEGAN 193
WORK COMPLETED 193
DELIVERED TO
REMARKS:

Tot. Hrs.
RATE
AMT.

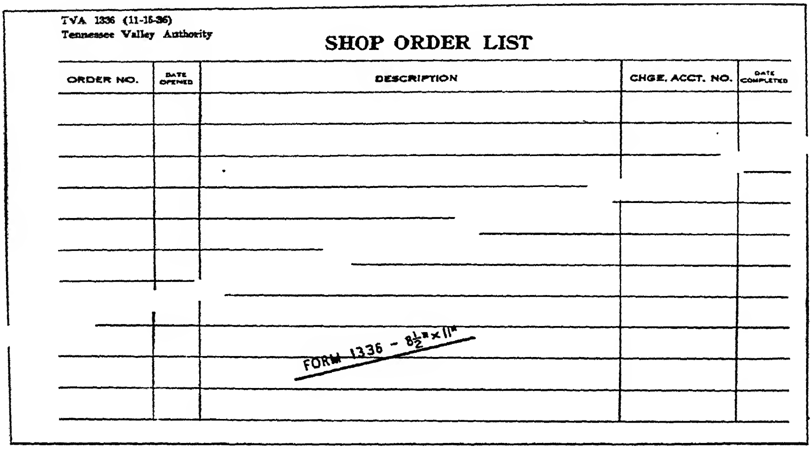
CR.

(SIGNED)
SHOP FOREMAN

FIGURE 306.—*Shop order.*

was obtained from daily memorandum records of shop employees' time. At the end of the month the labor and material costs were extended, totaled, and summarized. To this total was added a percentage to cover shop overhead and the new total charge posted to the proper account in the cost ledger through a journal voucher. All such orders were cleared each month, whether the work they covered was completed or not.

To facilitate record keeping, principal item of materials such as repair parts for trucks and tractors were charged where possible directly to the machine rather than through shop orders.

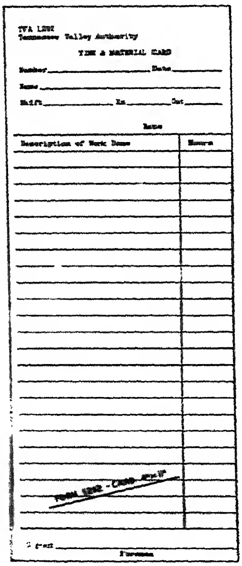

TVA 1336 (11-15-36)
Tennessee Valley Authority

SHOP ORDER LIST

ORDER NO.	DATE OPENED	DESCRIPTION	CHGE. ACCT. NO.	DATE COMPLETED

FORM 1336 - 8½"x11"

FIGURE 307.—*Shop order list.*


Tennessee Valley Authority

TIME & MATERIAL CARD

Description of Work Done | Hours

Foreman

FIGURE 308.—*Time and material card.*

Blanket shop orders.—Blanket shop orders were used for all small jobs performed in the shop rather than preparing a different order for each one. Upon receiving a request for work—as previously mentioned on a materials requisition—the job was entered on a shop work order list (fig. 307), and the item was numbered and classified by a member of the cost engineering staff who visited the shop daily for such purposes. Each shop employee kept a record of his time on a time and material card (fig. 308), recording the time spent and the material used on each job (indicated by number). From these cards were posted daily the labor and material charges to "accumulation sheets" of which, in any month, there was one for each account served. Each posting indicated the job item number, its description and purpose, and labor and material charges. At the end of each month, the completed major shop orders were sent with the month's "accumulation sheets" to the cost engineering office where blanket shop orders were prepared. The "accumulation sheets" were filed with other working papers in the cost engineering office, and the blanket shop order was turned over to the accounting office for vouchering.

Work chargeable to manufacturers.—Work was frequently performed by forces of the Authority for the benefit of manufacturers and subcontractors occasioned either by correcting errors of fabrication or as a facility for such contractors working on the job. Work orders were set up under the general coding of NDCW to cover work of this nature. No work was undertaken except on written authorization of a representative of the organization concerned. All charges were accumulated in the proper work order, and detailed invoices were prepared monthly for the signatures of the representatives.

Hired trucks and teams.

Throughout the construction period trucks and teams were hired to supplement the equipment of the Authority. They were paid for by the hour, their use being reported by foremen on excavation foreman's report (fig. 304) which was checked by the field time checkers. The owners' invoices were checked against these reports and then were handled through the receiving reports and later the invoice register, the same as material charges. The coding of such charges was made on the foreman's report to be later applied to the invoice when it was entered on the receiving report.

General expense and miscellaneous accounts.

General expenses consisted of administrative cost, engineering and superintendence, maintenance of headquarter facilities, project investigations, designing, and other items of administration which could not be equitably distributed as work progressed. This general expense is similar in nature to the clearing accounts in that it was distributed to the features upon completion, but was carried intact until the completion of the work. Monthly and interim cost reports were intended to reflect the total cost properly applicable to units of the work including the proper proportion of the various clearing accounts and the equipment and plant charges, but excluding any portion of the general expense or overhead.

The central office overhead applicable to all construction projects was distributed to the projects on the basis of labor. The project overhead, together with the central office overhead, was distributed to the feature accounts at the close of the job on the basis of total feature cost.

Land acquisition.—The account for the cost of land, as well as the cost of engineering and acquisition, was carried in a separate project account in the accounting office records in Knoxville.

Engineering planning and design.—The accounting for project investigation, design, and miscellaneous engineering services, which was done under the supervision of independent units of the Authority, was kept in the accounting office in Knoxville in a separate project account. The direct design charges by the United States Bureau of Reclamation were kept in the field accounting office.

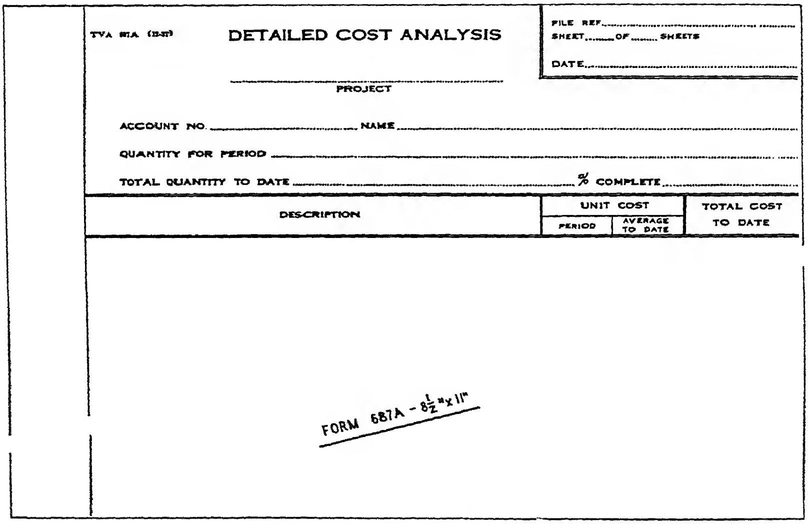
Camp and village.—No distribution was made of the cost of the camp and village operations to dam construction during the progress of the work, but an arbitrary amount based on total probable charge for this service was taken up monthly in the project books as an item of general expense with a corresponding credit to an account designated as "reserve for camp cost."

CONSTRUCTION COST ENGINEERING

Every effort was made by the cost engineers to develop significant cost information on controllable and salient construction features, and to bring it quickly to the attention of the superintendent and his staff. This information was developed in many ways.

Cost analyses.

Cost analyses were the principal means by which the cost engineers made information available for use either for job cost control, estimating, comparison with other jobs, or for record purposes. The analyses of any specific account involved the separating of the various



DETAILED COST ANALYSIS

FILE REF.
SHEET ____ OF ____ SHEETS
DATE

PROJECT

ACCOUNT NO. ____ NAME ____
QUANTITY FOR PERIOD ____
TOTAL QUANTITY TO DATE ____ % COMPLETE ____

DESCRIPTION	UNIT COST		TOTAL COST TO DATE
	PERIOD	AVERAGE TO DATE	

FORM 687A - $8\frac{1}{2}$" x 11"

FIGURE 309.—*Detailed cost analysis.*

components of costs found in the records, and the application of quantities furnished to obtain unit cost. They show total and unit costs of the different component trends of unit cost and important points that should be brought to the attention of the construction superintendent. Detailed analyses (fig. 309) were prepared for all principal features. For short periods of time, as conditions warranted, summarized analyses of the more salient construction features were prepared weekly from memorandum records and were not reconciled with the accounting records.

Certain salient recurrent features were analyzed monthly as a matter of routine, while other analyses were prepared especially either at the request of interested parties or when the advisability of an analysis was apparent.

It was necessary to make a close study of the information secured through analyses and to interpret the results. Occasional detection and correction of errors encountered gave greater significance to the

finished analysis. For example, in the case of concrete form analyses it was necessary to make adjustment for forms erected and not concreted, and for forms erected and concreted but not stripped in order that the unit cost for forms shown as a part of the concrete over-all unit cost would not be misleading.

Attempts were made to reduce costs into components which had definite significance and could be expressed in terms of its own quantities. For example, in the case of a concrete structure, the elements of cost were:

1. Cement and its handling, expressed in number of barrels of cement.
2. Coarse and fine aggregate, expressed in tons of each.
3. Forms, expressed in square feet of contact area.
4. Reinforcing steel and installation, expressed in weight of steel.
5. Finishing, expressed in area finished.
6. Concreting, expressed in volume of concrete placed, including mixing, hauling to the cableway, cableway handling, dumping and placing concrete into the form, cleaning and curing, and other miscellaneous operations.

In several instances (see fig. 193, for example) charts were kept showing the monthly variation of component unit costs, within a given total unit cost. Current cost analyses were given a fairly wide distribution. The superintendent and his assistants responsible for specified features were given copies in addition to copies distributed to the construction engineer and office executives.

Equipment plant and cost records.

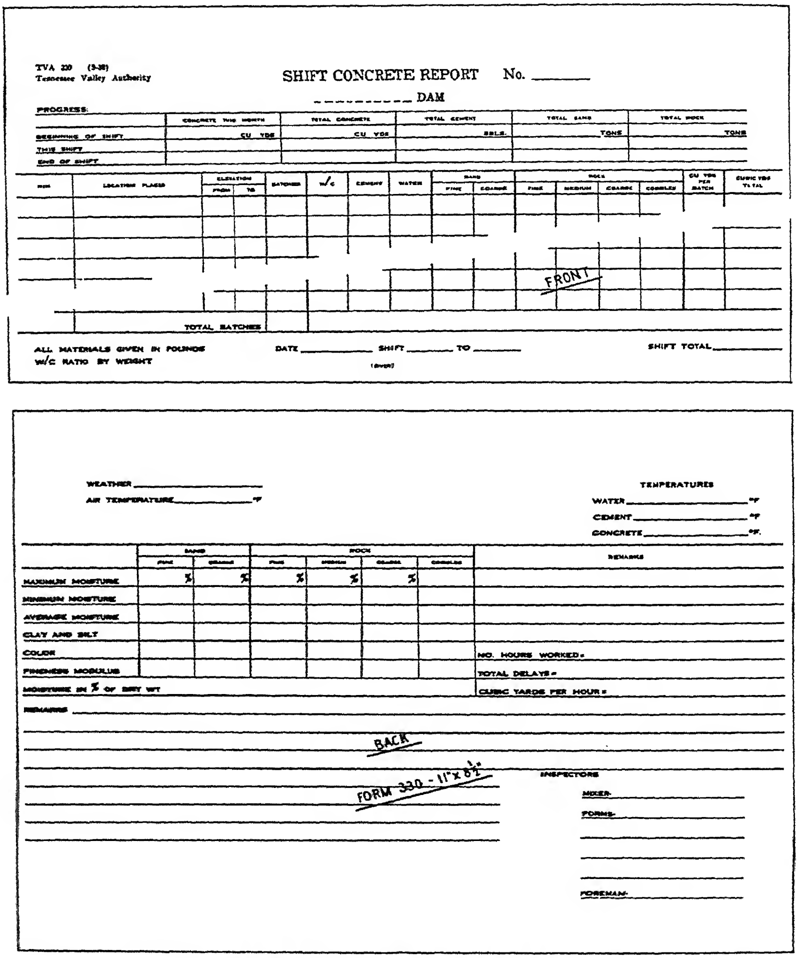
Aside from the objective of developing cost information which would enable the job authorities to control construction costs, it was necessary to leave a record of cost and equipment operations for other uses. It was realized that these records should be kept in such form that they would be available for a multiplicity of uses, such as:

1. Establishing probable cost of future projects or in determining their most economical design.
2. Comparing unit costs of Norris Dam with other projects of the Authority.
3. Setting up their permanent plant inventory record.
4. Preparing reports to the Federal Power Commission.

In general it was felt that where the conditions of the accounting records made it at all feasible, the expense and effort of establishing logical, accurate, and detailed statements of the components of all cost accounts were well worth while in that they would eliminate future expensive research. It was also realized that future research would not be as accurate or satisfactory as the analyses that could be compiled by the cost engineers during the construction period. In general, the analyses of feature accounts in the form established during the construction period were considered satisfactory for record purposes.

The procedure followed in establishing the final cost records was to prepare "preliminary final" analyses of cost accounts in balance with the accounting records before the final closure of the books, using such quantities as were available at the time. Narratives were prepared with the "preliminary final" analyses which were later used

with the final records. After the construction books were finally closed, the analysis of each account was carried forward from the "preliminary final" analyses, and adjustments were made when necessary for final costs.



TVA 330 (5-38)
Tennessee Valley Authority

SHIFT CONCRETE REPORT No. ______

__________ DAM

PROGRESS:

	CONCRETE THIS MONTH	TOTAL CONCRETE	TOTAL CEMENT	TOTAL SAND	TOTAL ROCK
BEGINNING OF SHIFT	CU YDS	CU YDS	BBLS.	TONS	TONS
THIS SHIFT					
END OF SHIFT					

MIX	LOCATION PLACED	ELEVATION		BATCHES	W/C	CEMENT	WATER	SAND		ROCK				CU YDS PER BATCH	CUBIC YDS TOTAL
		FROM	TO					FINE	COARSE	FINE	MEDIUM	COARSE	COBBLES		
	TOTAL BATCHES														

ALL MATERIALS GIVEN IN POUNDS
W/C RATIO BY WEIGHT

DATE ____ SHIFT ____ TO ____ SHIFT TOTAL ____

(over)

FRONT

WEATHER ______
AIR TEMPERATURE ____ °F

TEMPERATURES
WATER ____ °F
CEMENT ____ °F
CONCRETE ____ °F.

	SAND		ROCK				REMARKS
	FINE	COARSE	FINE	MEDIUM	COARSE	COBBLES	
MAXIMUM MOISTURE	%	%	%	%	%		
MINIMUM MOISTURE							
AVERAGE MOISTURE							
CLAY AND SILT							
COLOR							NO. HOURS WORKED =
FINENESS MODULUS							TOTAL DELAYS =
MOISTURE IN % OF DRY WT							CUBIC YARDS PER HOUR =

REMARKS ______

BACK

FORM 330 - 11" x 8½"

INSPECTORS
MIXER-
FORMS-
FOREMAN-

FIGURE 310.—*Shift concrete report.*

Cost estimates.

The purposes of estimates made by the cost engineers were varied. Principally the estimates were in connection with proposed alternate design or construction methods and with proposed modification of the construction schedule. The most frequent work of this nature

was in connection with the construction budget and monthly comparison of actual and revised estimated cost with the budget, and with the original estimate. The work of preparing such estimates was usually assigned to the cost engineers because they were most familiar with unit costs and the conditions affecting them.

TVA 322
Tennessee Valley Authority

NORRIS DAM

FOUNDATION INSPECTOR'S SHIFT REPORT

Date ________
Shift ________

Inspector: ____________________

DRILLING

Equipment	Location	Lin. Ft. (Shift)	Total Depth	No. Holes	Remarks
36" Calyx					
5-1/2" Calyx					
___(Wagon)					

FORM 322 - 8½"x11"

WASHING SEAMS

Location	Remarks

GROUTING

Location	Cement	Remarks

FIGURE 311.—*Norris Dam foundation inspector's shift report.*

Daily reports for job control.

Items of particular significance were submitted daily to the superintendent, information concerning 1 day's operations being reported on the next. Such reports included progress made, labor gang operations, labor classifications, and pay roll charges to cost account and

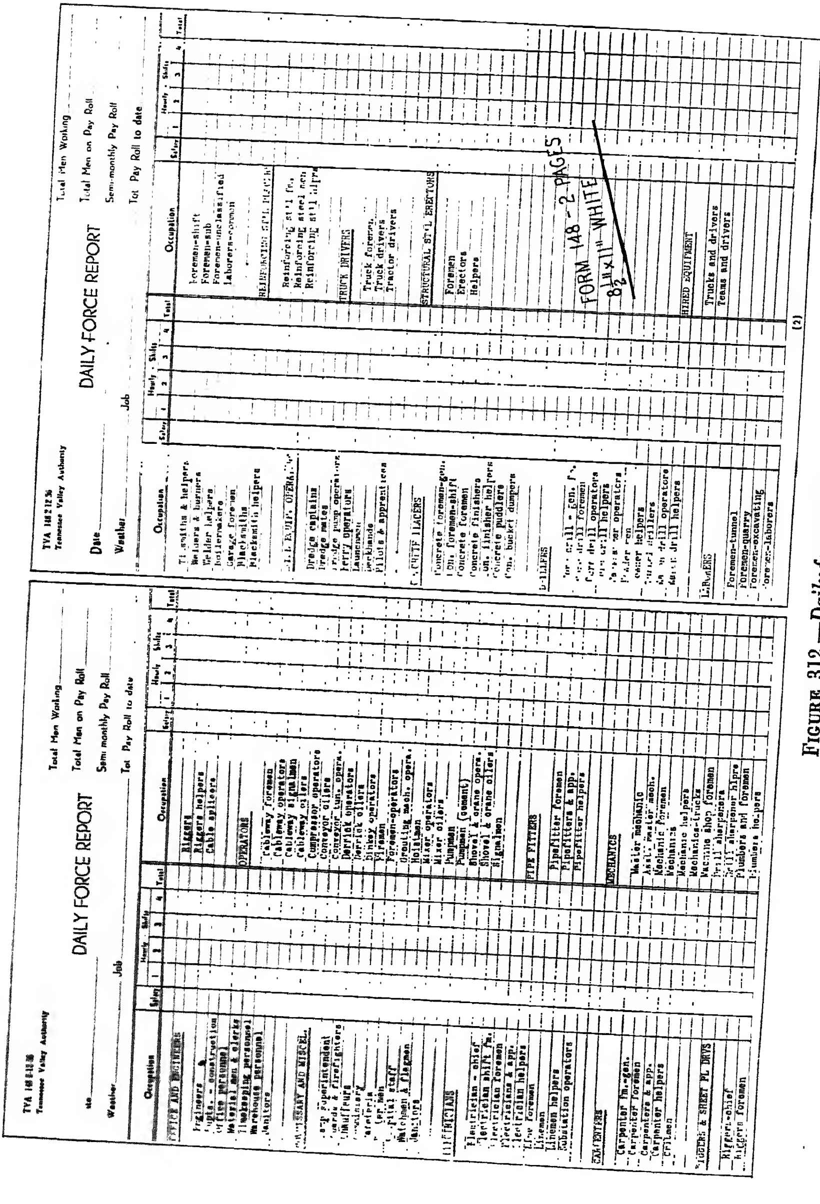
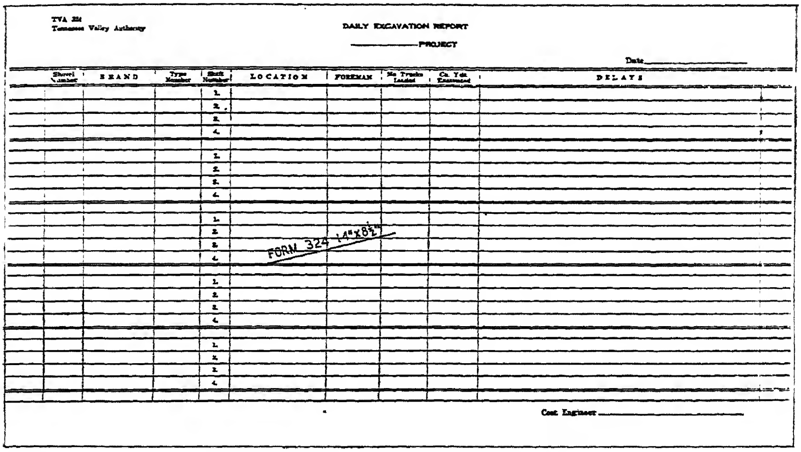


FIGURE 312.—*Daily force report.*

were supplemented at times by partial cost figures on construction items. While all such reports did not originate with the cost engineering staff, they were prepared with the ultimate view of controlling and reducing job costs. Concrete progress for the day was recorded on the shift concrete report (fig. 310) prepared by the concrete technologist for each shift on which the mixing plant was operated. A foundation inspector's shift report (fig. 311) was turned in by each drilling and grouting inspector stating progress of drilling and grouting for foundation treatment. Daily force reports (fig. 312) were prepared by the time office showing the number of men on the job in each classification, the number of absentees, and the men hired and terminated (also by classification). These were supplemented by reports, also prepared by the time office but at less frequent intervals, showing the foreman of each gang by name, the number of men


TVA 324
Tennessee Valley Authority

DAILY EXCAVATION REPORT

________ PROJECT

Date ________

Shovel Number	Brand	Type Number	Shift Number	Location	Foreman	No. Trucks Loaded	Cu. Yds. Excavated	Delays
			1.					
			2.					
			3.					
			4.					
			1.					
			2.					
			3.					
			4.					
			1.					
			2.					
			3.					
			4.					
			1.					
			2.					
			3.					
			4.					
			1.					
			2.					
			3.					
			4.					

FORM 324 14"x8½"

Cost Engineer ________

FIGURE 313.—*Daily excavation report.*

of each classification in his crew, and the principal activity reported for the day, with a recapitulation, by shifts, of men employed, showing their rates. Daily excavation reports (fig. 313) were prepared by the cost engineers showing the location, foreman, progress, and delays of each shovel. Labor cost comparisons (fig. 314) were also prepared on which were tabulated daily the total direct labor charges to each account which was listed by number and description. This report provided space for 8 days so that it was possible to compare the labor charges of an account for an entire week with the last day or average of the preceding week. Other special daily progress reports which were prepared when needed included individual operations of short duration but which warranted close supervision, such as excavation and concreting in the east abutment tunnel and core wall, riveting the scroll cases, and erection and riveting of the powerhouse structural steel framework.

TVA [illegible]
Tennessee Valley Authority

LABOR COST COMPARISON

SHEET________

FOR SEVEN CONSECUTIVE DAYS

__________ PROJECT

FROM________ TO________

ACCOUNT NUMBER	DESCRIPTION OF ACCOUNT	LAST DAY OF PREVIOUS WK.	DATE	DATE	DATE	DATE	DATE	DATE	DATE	SEVEN DAY TOTAL	REMARKS UNITS, QUANTITIES, UNIT COSTS, ETC.

FORM 1323 - 17" x 11"

This report is designed primarily to give the Construction Superintendent a daily report of the distribution of labor. Each day the new charges are entered in the proper spaces. Accounts should be grouped according to physical location.

FIGURE 314.—*Labor cost comparison.*

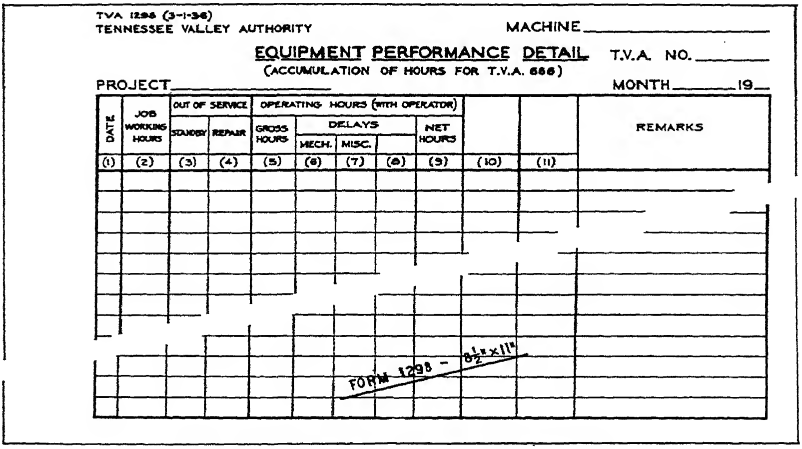
Time studies.

Detailed studies of certain construction operations where opportunities for considerable savings appeared to exist were made by a field time study crew of three men. The first such study made covered the operating time, delays, and progress on wagon drilling in the quarry and in foundation excavations. Another study covering a 2-month period was made on the production, operating time, and delays in the operation of the coarse and fine aggregate plant. The most lengthy study undertaken was in the quarry in connection with drill bits. This study covered a period of 5 months, during which time a comparative test was run of the performance of regular drill steel and three makes of detachable bits for the wagon drills (see p. 230). Another major assignment was a detailed time and production study of panel form operations for mass concrete. In this study, relative costs of the various component operations were established, and certain recurrent causes of increased cost were brought to light. Subsequent assignments took the form of shorter studies on structural steel erection and riveting and scroll case riveting.

Such time studies developed information which was not only valuable from the standpoint of assisting in controlling job costs but was useful in developing more highly refined procedures in selections of better equipment at other projects of the Authority.

Machine operations and delays.

On principal pieces of the construction equipment and on principal parts of the assembled plants, a record was kept of the delays as well as the operating time (fig. 315). Delays were subdivided into me-



TVA 1298 (3-1-36)
TENNESSEE VALLEY AUTHORITY

MACHINE________

EQUIPMENT PERFORMANCE DETAIL T.V.A. NO.________
(ACCUMULATION OF HOURS FOR T.V.A. 666)

PROJECT________ MONTH________19__

DATE	JOB WORKING HOURS	OUT OF SERVICE		OPERATING HOURS (WITH OPERATOR)							REMARKS
		STANDBY	REPAIR	GROSS HOURS	DELAYS			NET HOURS			
					MECH.	MISC.					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	

FORM 1298 - $8\frac{1}{2} \times 11$"

FIGURE 315.—*Equipment performance detail.*

chanical and miscellaneous delays. When summarized, such information indicated weaknesses in operation which might be corrected. It also provided valuable information for use in making future purchases of equipment and plant for other projects. Delay studies which re-

ceived the most job consideration were those of assembled plants such as the cableway, mixing plant, sand plant (fig. 316), and crushing and screening plant (fig. 317).

TVA 669
Tennessee Valley Authority

SAND PLANT

Norris Dam

OPERATION REPORT

Date: ____________
Shift: ____________
Foreman: ____________

Shutdowns

Duration Hrs	Duration Min	Cause
___	___	____________________
		____________________
___	___	____________________
		____________________
___	___	____________________
___	___	____________________
		____________________

Lost Time - Screens

Screen No.	Hrs	Min	Cause
___	___	___	____________________
___	___	___	____________________

New Screen Cloth

Screen No.	Size Opening
___	___
___	___

	A-C Mills Hrs	A-C Mills Min	Penna. Mills Hrs	Penna. Mills Min
No. 1	___	___	___	___
No. 2	___	___	___	___
H.M. Hrs.	___	___	___	___

Hammer Changes

	Kind of Change	Time	Time Required
A-C – 1	____________	___	___
A-C – 2	____________	___	___
Penn– 1	____________	___	___
Penn– 2	____________	___	___

REMARKS: ____________________

FORM 669 - $8\frac{1}{2}$" × 11"

FIGURE 316.—*Sand plant operation report.*

Repair and replacement analyses.

Careful studies were made of repair and replacement items on major pieces of equipment and assembled plants. The master mechanic maintained a log on form 665 (fig. 318) of the replacement parts for major pieces of equipment. This log disclosed weaknesses in the equipment which required frequent repair, and was of considerable assistance in making future purchases.

Detailed records were kept on the replacements of screens in the screening plant and on the replacement of hammers for the sand plant hammer mills. Information disclosed in these records resulted in great economy by the selection of parts which gave longer life.

TVA 670
Tennessee Valley Authority

CRUSHING AND SCREENING PLANT

Norris Dam

OPERATION REPORT

Date: ______
Shift: ______
Foreman: ______

Shutdowns

Duration Hrs	Duration Min	Cause
___	___	______
___	___	______
___	___	______
___	___	______
___	___	______

Lost Time - Screens

	Hrs	Min	Cause
#1	___	___	______
2	___	___	______
3	___	___	______
4	___	___	______
5	___	___	______
6	___	___	______
7	___	___	______
8	___	___	______

New Screen Cloth

Screen No.	Diameter of Wire	Size Opening
______	______	______
______	______	______

FORM 670 - 8½" x 11"

REMARKS: ______

FIGURE 317.—*Crushing and screening plant operation report.*

CONSTRUCTION BUDGET

The general procedure under which funds were made available for the construction was that the construction organization prepared semiannual estimates of the funds required to perform construction in accordance with the approved construction schedule. The esti-

mates were submitted to the assistant chief engineer in Knoxville, where all such similar requests from other projects were assembled, correlated for omission and overlapping, and a final statement prepared showing the needs for all engineering and construction operations. This statement was then submitted to the general manager and in turn to the Board of Directors for approval. When funds were appropriated by Congress the amounts as shown in the approved budgets were earmarked to each organization.

TVA 665 11-1-36

TENNESSEE VALLEY AUTHORITY
EQUIPMENT REPORT
OF
REPAIR AND MECHANICAL CONDITION

PROJECT ________ ITEM ________ FROM ________ TO ________ SHEET ________ OF ________

ASSEMBLY LOCATION	DATE	SERVICE OBTAINED TO DATE	BRIEF DESCRIPTION OF NEW PART OR REPAIR JOB	PART NUMBER	APPROX. COST	REASON FOR REPAIR

INCLUDE STATEMENT OF MECHANICAL CONDITION--USE REVERSE SIDE IF NECESSARY

FORM 665 - 11"x8½"

FIGURE 318.—*Equipment report of repair and mechanical condition.*

The expenditures of each organization were recorded by the central budget office and a running record was kept of obligations as compared to funds allotted. A similar check was maintained in the field office. In general, when funds were allocated to a given organization for an operation which included several elements, no attempt was made to force the organization to allot its funds between the different elements exactly as originally estimated. Within an organizational budget, therefore, overruns were balanced against underruns. However, organization budgets were watched carefully to see that obligations were not incurred in excess of allotted amounts, and that allotments in excess of requirements were released to the unallotted general fund of the Authority. Unobligated balances of allotments at the end of a fiscal year were transferred to the general fund.

Budget requests.

Budget requests consisted essentially of a brief statement of the scope and estimated cost of the work to be done under the requested allotment, a summarized cost statement by construction features showing obligations to the beginning of the fiscal period, estimated obligations for the fiscal period, estimated obligations subsequent to the fiscal period, total estimated obligations, and the amount of the original approved preliminary cost estimates. Supplementing this, there was attached a list of salaried employees by name, rate, and

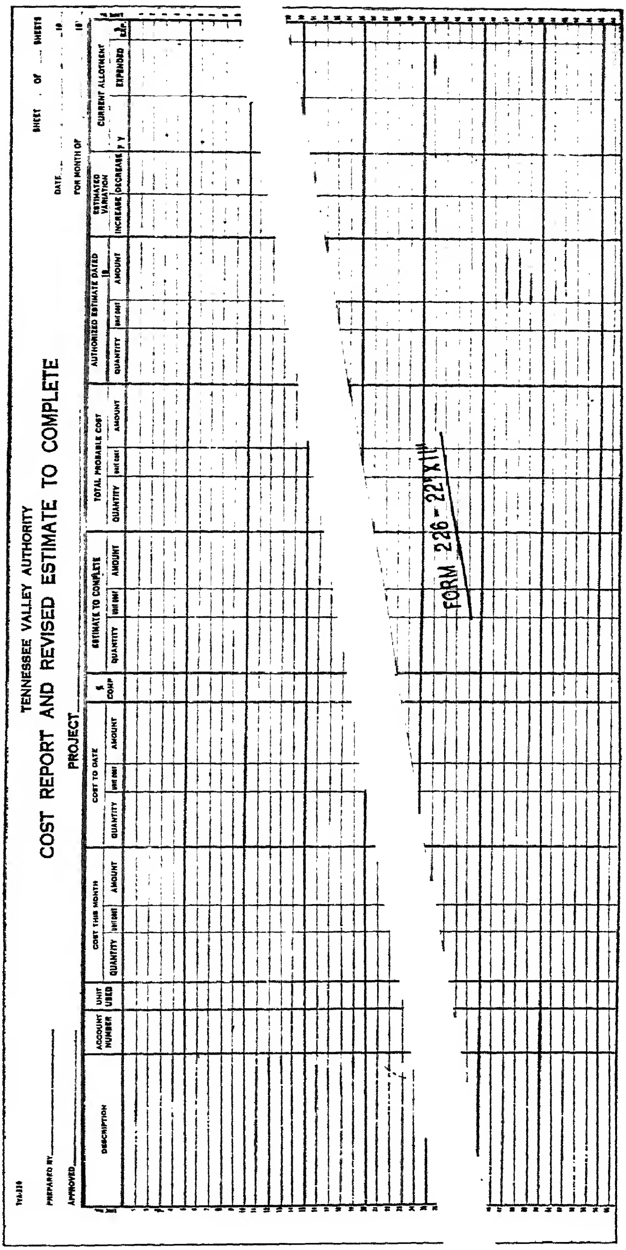


FIGURE 319.—*Cost report and revised estimate to complete.*

anticipated earnings: a list of the estimated average number of hourly employees in each classification, and their anticipated earnings; and a summarized statement of the expected cost of the work to be done directly or indirectly by the field office.

Budget estimates were in general submitted semiannually, in time for consideration before June 30 (the end of the fiscal year) and January 1 (the middle of the fiscal year). Based on the June 30 estimate, the allotment for the entire coming fiscal year was made. At the midyear the allotment was modified, as indicated by the more recent estimates.

Monthly budget report.

The budget office in Knoxville issued a monthly report, without detail, comparing the status of obligations against individual allotments, with the allotments. In order to give a check of the budgetary status for the job, a more detailed monthly report was used in which it was possible to use the more recently developed unit costs and estimates of uncompleted quantities. This made it possible to forecast probable modifications of previously estimated obligations in the light of more recent developments on the job. Such a procedure was of great value to the job officials in checking the status of the project with respect to budgetary consideration and to the approved preliminary cost estimates.

The form used for this report was TVA 226 (fig. 319), cost report and revised estimate to complete. Money amounts for the month and to date were secured from the accounting records. Quantities for the month, to date, and estimated to complete the project were secured from the engineering organization. Extensions of the revised estimates to complete and the revised estimated total costs were made by the cost engineer, being guided not only by the unit cost developed, but by all other considerations bearing on anticipated future costs which could be determined, such as the increased ratio of formed area to concrete volume in the dam as work progressed from the base to the crest, and probable increases in unit costs due to the tapering off of heavy construction.

SUMMARY OF COSTS

The final cost of the original Norris project as reported herein at $32,269,027.13 (General Summary, Table 100–A), includes hydraulic multiple-purpose plant (consisting of land and land rights for which title rests in the United States Government, structures, improvements, and equipment), together with transmission plant and general plant constructed concurrently with the dam. The construction cost of the town of Norris, forest improvements to reservoir lands, Big Ridge and Norris Parks, the service building, the fish hatchery, and other similar nonproject items are not reflected in this report.

The final cost of the project, by character of expenditures, is as follows (Detailed Summary, Table 100–B):

Land costs		$8,740,240.12
Construction costs:		
Direct construction costs	$18,584,745.37	
Indirect construction costs	1,741,590.41	
		20,326,335.78

Distributive general expenses:		
Design and construction engineering	$1,827,494.40	
Executive and administrative costs	1,057,421.51	
Other general costs	317,535.32	
		$3,202,451.23
Total cost		32,269,027.13

Land costs in the amount of $8,740,240.12 cover reservoir and Norris freeway land purchased in fee, flowage easements and highway relocation easements, for which title rested in the United States Government as of September 30, 1938.

Direct construction costs, totaling $18,584,745.37, are for labor, material, construction plant, equipment, tools, shop expense, warehouse charges, transportation, and other similar expenditures.

Indirect construction costs total $1,741,590.41 and include field superintendence, field office expense, provision for medical service, police and guide service, and the cost of camp operation including the entire first cost of temporary camp facilities and normal depreciation during the construction period on the permanent town facilities (p. 631). These costs have been distributed on the basis of the direct construction costs of those organizations incurring them.

Design and construction engineering costs amounting to $1,827,494.40 include the Norris project's proportion of the chief engineer's general administrative office, the salaries and expenses of executive and supervisory engineers, cost engineers, concrete technicians, inspectors, engineering for control lines, geologic studies, design of structures including the United States Bureau of Reclamation design charges and a small amount of consultation fees (p. 632). These engineering costs have been distributed on the basis of related construction costs.

Executive and administrative costs, totaling $1,057,421.51, include the Norris project's proportion of the salaries and expenses of the general administrative offices of the Authority and those division administrative offices, such as the construction and maintenance and the highway and railroad organizations that were concerned with the construction of specific portions of the project (p. 632). These costs have been distributed likewise on the basis of related construction costs.

Other general costs of $317,535.32 include such charges as reservoir and dam site surveying and mapping, hydraulic studies, and general project planning (p. 632). These have been distributed over all accounts exclusive of land costs.

In the "Three-Plant Allocation Report," [1] as approved on June 6, 1938, a preliminary figure was used for Norris hydraulic multiple-purpose plant of $31,532,120. This was based upon the books of account as of February 28, 1938, adjusted to exclude those items of cost not subject to allocation and to include estimates of allocable expenditures yet to be made.

The balance sheet of the Authority of June 30, 1938, shows the hydraulic multiple-purpose plant of the Norris project at $30,749,776.34, a reduction of $782,344, which is accounted for by elimination of such items as estimates to complete, miscellaneous structures (fish

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[1] See appendix H.

hatchery, service building, and forestry plant), a portion of the camp loss resulting from a redetermination, and forestry work.

The cost of the hydraulic multiple-purpose plant as reported herein, totaling $30,508,024.41 (table 100–A), differs from the June 30, 1938, balance sheet figure principally because of the elimination of land costs, where clear title has not yet passed to the United States, and the elimination of land costs not related to the reservoir, the Norris freeway and highway relocations.

The Authority has deposited $139,546.43 with the courts for land in condemnation. As the cases are settled and clear title has passed to the United States, the final costs thereof including acquisition and condemnation costs will be added to the project cost.

The schedules following represent summary statements of detailed analyses of cost.

TABLE 100–A.—*Final project cost—general summary*

Account	Description	Amount
	HYDRAULIC MULTIPLE PURPOSE PLANT	
20	Land and land rights	$12,992,707.94
21	Structures and improvements	1,632,051.90
22	Reservoirs, dams, and waterways	12,948,564.08
23	Water wheels, turbines, and generators	2,095,826.87
24	Accessory electric equipment	317,932.91
25	Miscellaneous power plant equipment	245,040.37
26	Roads, railroads, and bridges	275,900.34
	Total	30,508,024.41
	TRANSMISSION PLANT	
42	Structures and improvements	26,253.53
43	Station equipment	966,164.41
	Total	992,417.94
	GENERAL PLANT	
	General plant—Norris	753,248.97
	General plant—other	15,335.81
	Total	768,584.78
	Grand total	32,269,027.13

TABLE 100-B.—*Final project cost—detailed summary*

		Land costs	Direct construction costs: labor, material, and other charges	Indirect construction costs: Norris freeway distribution	Indirect construction costs: Camp and other indirect	Total direct and indirect construction costs	Distributive general expense: Design and construction engineering	Distributive general expense: Executive and administrative costs	Distributive general expense: Other general costs	Total
	HYDRAULIC MULTIPLE PURPOSE PLANT									
	Land and land rights:									
200	Purchase price of land	$7,648,271.54				$7,648,271.54				$7,648,271.54
201	Expense of land and privilege acquisition	951,857.15				951,857.15				951,857.15
204	Relocating highways and highway bridges	117,164.60	$2,031,911.70		$76,830.70	2,225,907.00	$334,702.52	$94,385.50	$34,705.22	2,689,700.24
205	Relocating railways and railway bridges		992,959.41		23,702.71	1,016,662.12	118,743.12	26,399.71	15,887.86	1,177,692.81
207	Relocating other structures and improvements		280,570.87		8,836.93	289,407.80	58,247.26	11,591.65	4,912.75	364,159.46
208	Protecting reservoir property		78,873.57			78,873.57			1,078.61	79,952.18
209	Protecting existing structures and improvements		67,217.40		4,436.48	71,653.88	4,445.08	3,881.85	1,093.75	81,074.56
		8,717,293.29	3,451,532.95		113,806.82	12,282,633.06	516,137.98	136,258.71	57,678.19	12,992,707.94
	Structures and improvements:									
211	General preparation of site		3,966.68	$192.12	454.17	4,612.97	349.14	225.84	70.95	5,258.90
212	General yard improvements		146,259.14	5,062.72	13,712.70	165,034.56	10,400.65	8,108.40	2,509.99	186,053.60
213	Powerhouse		1,072,820.80	51,960.40	122,834.31	1,247,615.51	94,426.48	61,081.82	19,187.94	1,422,311.75
217	Air, water, and automotive terminals		13,899.60	673.21	1,591.46	16,164.27	1,223.40	791.38	248.60	18,427.65
			1,236,946.22	57,888.45	138,592.64	1,433,427.31	106,399.67	70,207.44	22,017.48	1,632,051.90
	Reservoirs, dams, and waterways:									
220	Reservoir		1,427,946.70	13,928.90	123,341.39	1,565,216.99	84,142.27	176,931.51	24,974.81	1,851,265.58
221	Concrete dam and spillway		7,755,118.59	375,607.07	887,934.49	9,018,660.15	682,582.35	441,543.25	138,704.17	10,281,489.92
223	East embankment		380,383.78	18,423.29	43,552.65	442,359.72	33,480.24	21,657.42	6,803.35	504,300.73
224	Buffalo Creek divide dike		80,377.15	3,892.94	9,202.91	93,473.00	7,074.56	4,576.33	1,437.59	106,561.48
226	Water conductors		154,586.88	7,487.17	17,699.67	179,773.72	13,606.27	8,801.52	2,764.86	204,946.37
			9,798,413.10	419,339.37	1,081,731.11	11,299,483.58	820,885.69	653,510.03	174,684.78	12,948,564.08
	Water wheels, turbines, and generators:									
230	Foundations and miscellaneous steel and iron		3,586.80	173.72	410.67	4,171.19	315.70	204.22	64.15	4,755.26
231	Turbines, including scroll case, speed ring, and draft tube liners		586,809.27	28,425.55	67,197.96	682,522.78	51,657.12	33,415.53	10,496.99	778,092.42

TABLE 100–B.—*Final project cost—detailed summary*—Continued

No.	Item	Land costs	Direct construction costs: labor, material, and other charges	Indirect construction costs: Norris freeway distribution	Indirect construction costs: Camp and other indirect	Total direct and indirect construction costs	Distributive general expense: Design and construction engineering	Distributive general expense: Executive and administrative costs	Distributive general expense: Other general costs	Total
	HYDRAULIC MULTIPLE PURPOSE PLANT—continued									
	Water wheels, turbines, and generators—Continued.									
232	Auxiliary equipment for turbines	----------	$39,514.28	$1,913.81	$4,524.26	$45,952.35	$3,477.93	$2,240.77	$706.73	$52,386.78
235	Generators	----------	933,764.71	45,225.44	106,912.85	1,085,903.00	82,187.18	53,164.57	16,700.85	1,237,955.60
236	Auxiliary equipment for generators	----------	7,744.55	375.10	886.73	9,006.38	681.65	440.94	138.52	10,267.49
239	Miscellaneous equipment for turbines and generators	----------	9,329.93	451.88	1,068.24	10,850.05	821.19	531.21	160.87	12,369.32
		----------	1,580,839.54	76,565.50	181,000.71	1,838,405.75	139,140.77	90,006.24	28,274.11	2,095,826.87
	Accessory electric equipment:									
241	Switchgear	----------	19,845.81	961.20	2,272.27	23,079.28	1,746.77	1,129.94	354.95	26,310.94
242	Switchboards	----------	110,270.54	5,340.78	12,625.62	128,236.94	9,705.68	6,278.33	1,072.24	146,193.19
243	Protective equipment	----------	9,608.84	465.39	1,100.19	11,174.42	845.74	547.09	171.86	12,739.11
244	Electrical structures	----------	9,718.07	470.68	1,112.68	11,301.43	855.36	553.31	173.81	12,883.91
245	Conduit work	----------	37,057.19	1,794.81	4,242.92	43,094.92	3,261.66	2,109.88	662.78	49,129.24
246	Power wiring	----------	13,502.72	653.98	1,546.01	15,702.71	1,188.47	768.79	241.50	17,901.47
247	Control wiring	----------	8,130.25	393.78	930.89	9,454.92	715.60	462.90	145.41	10,778.83
249	Station service equipment	----------	31,676.89	1,534.22	3,626.89	36,838.00	2,788.11	1,803.55	566.56	41,996.22
		----------	239,810.31	11,614.84	27,457.47	278,882.62	21,107.39	13,653.79	4,289.11	317,932.91
	Miscellaneous power plant equipment:									
252	Station maintenance equipment	----------	12,797.04	619.81	1,465.22	14,882.07	1,126.36	728.61	228.88	16,965.92
255	Cranes and hoisting equipment	----------	75,929.74	3,677.54	8,693.71	88,300.99	6,683.11	4,323.11	1,358.04	100,665.25
256	Compressed air and vacuum cleaning systems	----------	8,552.04	414.20	979.18	9,945.42	752.73	486.92	152.96	11,338.03
258	Station service water system	----------	12,183.86	590.11	1,395.00	14,168.97	1,072.39	693.70	217.92	16,152.98
259	Other miscellaneous equipment	----------	75,366.26	3,650.25	8,629.18	87,645.69	6,633.51	4,291.03	1,347.96	99,918.19
		----------	184,828.94	8,951.91	21,162.29	214,943.14	16,268.10	10,523.37	3,305.76	245,040.37
	Roads, railroads, and bridges:									
260	Access roads for permanent use	----------	49,168.20	2,381.39	5,629.59	57,179.18	4,327.64	2,799.43	879.40	65,185.65
262	Roadway on concrete dam	----------	158,937.80	7,697.91	18,197.83	184,833.54	13,989.23	9,049.24	2,842.68	210,714.69
		----------	208,106.00	10,079.30	23,827.42	242,012.72	18,316.87	11,848.67	3,722.08	275,900.34
	Total hydraulic multiple purpose plant	$8,717,293.29	16,700,477.06	584,439.37	1,587,578.46	27,589,788.18	1,638,256.47	986,008.25	293,971.51	30,508,024.41
	TRANSMISSION PLANT									
	Structures and improvements:									
421	General preparation of site	----------	12,322.87	596.84	1,410.93	14,330.64	1,084.62	701.61	220.40	16,337.27
422	General yard improvements	----------	7,479.63	362.26	856.40	8,698.29	658.33	425.86	133.78	9,916.26
		----------	19,802.50	959.10	2,267.33	23,028.93	1,742.95	1,127.47	354.18	26,253.53
	Station equipment:									
430	Outdoor substation structure	----------	113,454.83	5,495.01	12,990.20	131,940.04	9,985.96	6,459.63	2,029.20	150,414.83
431	Switchgear	----------	217,209.13	10,520.19	24,869.71	252,599.03	19,118.10	12,366.96	3,884.89	287,968.98
433	Protective equipment	----------	28,551.25	1,382.84	3,269.02	33,203.11	2,513.00	1,625.59	510.65	37,852.35
435	Conduit work	----------	8,752.91	423.93	1,002.19	10,179.03	770.40	498.35	156.55	11,604.33
436	Power wiring	----------	45,521.06	2,204.74	5,212.01	52,937.81	4,006.63	2,591.77	814.17	60,350.38
437	Control wiring	----------	9,288.85	449.89	1,063.54	10,802.28	817.58	528.87	166.14	12,314.87
438	Main conversion equipment	----------	286,821.90	13,891.77	32,840.13	333,553.80	25,245.21	16,330.41	5,129.95	380,259.37
439	Station service equipment	----------	19,158.17	927.90	2,193.56	22,279.63	1,686.24	1,090.78	342.65	25,399.30
		----------	728,758.10	35,296.27	83,440.36	847,494.73	64,143.12	41,492.36	13,034.20	966,164.41
	Total transmission plant	----------	748,560.60	36,255.37	85,707.69	870,523.66	65,886.07	42,619.83	13,388.38	992,417.94
	GENERAL PLANT									
	Norris:									
	Norris freeway from highway, Tennessee 33, to West Norris Road	12,931.36	610,642.32	----------	35,497.07	659,070.75	68,217.31	15,973.53	9,987.38	753,248.97
	Other:									
	Norris freeway from highway U. S. 25–W to Coal Creek yard	1,396.88	12,463.57	----------	----------	13,860.45	1.287.31	----------	188.05	15,335.81
	Total general plant	14,328.24	623,105.89	----------	35,497.07	672,931.20	69,504.62	15,973.53	10,175.43	768,584.78
		8,731,621.53	18,072,143.55	620,694.74	1.708,783.22	29,133,243.04	1,773,647.16	1,044,601.61	317,535.32	32,269,027.13
	Indirect construction costs—Norris freeway classified by character of expenditures:									
	Land costs	8,618.59	----------	[1] 8,618.59	----------	----------	----------	----------	----------	----------
	Direct construction costs	----------	512,601.82	[1] 512,601.82	----------	----------	----------	----------	----------	----------
	Indirect constructions costs	----------	----------	[1] 32,807.19	32,807.19	----------	----------	----------	----------	----------
	Design and construction engineering	----------	----------	[1] 53,847.24	----------	[1] 53,847.24	53,847.24	----------	----------	----------
	Executive and administrative costs	----------	----------	[1] 12,819.90	----------	[1] 12,819.90	----------	12,819.90	----------	----------
	Totals	8,740,240.12	18,584,745.37	----------	1.741,590.41	29.066,575.90	1.827,494.40	1,057,421.51	317,535.32	32.269,027.13

[1] Denotes credit.

TABLE 100-C.—*Final project cost—details*

LAND COSTS AND DIRECT CONSTRUCTION COSTS

HYDRAULIC MULTIPLE-PURPOSE PLANT

LAND AND LAND RIGHTS

Account	Description	Quantity	Unit	Rate	Amount
200	Purchase price of land:				
-1	Reservoir land—Fee purchases. Excludes tracts in process of condemnation where title had not been transferred as of Sept. 30, 1938:				
-10	Purchased under resolution of board of directors, dated Oct. 16, 1933, authorizing acquisition of all necessary tracts lying within ¼ mile of flood stage (elevation 1047) (2,191 tracts).	110,809.64	Acre	$58.82	$6,517,867.65
-11	Purchased under separate board resolutions, authorizing acquisition of Central Peninsula area, small isolated peninsulas, and other lands within the reservation limits (543 tracts).	30,960.42	Acre	36.42	1,127,543.89
-2	Reservoir land rights—Flowage. Easement purchases (5 tracts).	36.18	Acre	79.05	2,860.00
	Total account No. 200	141,806.24	Acre	53.93	7,648,271.54
201	Expense of land and privilege acquisition: Cost of acquiring land and land rights set forth in account 200 (141,806.24 acres).	2,739	Tract	347.52	951,857.15
	Total account No. 201				951,857.15
204	Relocating highways and highway bridges:				
-0	State highways:				
-01	Highway No. 33 including drainage structures.				475,694.77
-02	Highway No. 9 including bridges and drainage structures.				118,827.11
-03	Highway No. 63 including drainage structures.				57,047.71
-04	Highway No. 32 including drainage structures.				16,953.46
-05	Clinch River Bridge				349,795.04
	Total State highways	11.97	Mile	85,072.52	1,018,318.09
-1	County highways:				
-100	Peninsula Rd. including bridges and drainage structures.				86,817.45
-101	Wells Spring Rd. including drainage structures.				26,164.00
-102	Agee-LaFollette Rd. including drainage structures.				166,974.43
-103	Big Sink-Stiner Rd. including bridges and drainage structures.				12,889.48
-104	Hickory Valley Rd. including drainage structures.				24,887.39
-105	Lone Mountain Rd. including drainage structures.				132,177.85
-106	Flat Hollow Rd. including bridges and drainage structures.				19,696.08
-107	Lindsay Mill Rd. including drainage structures.				15,756.41
-108	Primroy Rd. including drainage structures.				18,216.69
-109	Whitman Hollow Rd. including drainage structures.				22,453.04
-110	Ruben Hutson Rd. including drainage structures.				2,948.93
-111	Cove Creek Rd. including drainage structures.				2,565.46

TABLE 100-C.—*Final project cost—details*—Continued

LAND AND LAND RIGHTS—Continued

Account	Description	Quantity	Unit	Rate	Amount
204	County highways—Continued.				
-112	Black Fox Creek Rd. including drainage structures.				$53,291.51
-113	Log Mountain Rd. including drainage structures.				16,790.23
-114	Cracker Creek Rd. including bridge and drainage structures.				42,363.82
-115	Alternate for Highway No. 61 including drainage structures.				15,095.78
-116	Henry Store Rd. including bridge and drainage structures.				34,800.50
-117	Indian Creek Rd. including drainage structures.				20,032.67
-118	Arnwine Rd. including drainage structures.				29,250.32
-119	Pine Crest Rd. including drainage structures.				73,728.65
-120	County connections highway No. 33 at Big Barren including drainage structures.				5,374.54
-121	Andersonville-Loyston Rd.				1,268.74
-122	Primroy Bridge				29,514.74
-123	Big Creek Bridge				24,820.17
	Total county highways	64.5	Mile	$13,610.53	877,878.88
-2	Tertiary roads including bridges and drainage structures.	23.79	Mile	3,585.38	85,296.24
-3	Bridge removals	35	Bridge	759.52	26,583.09
-4	Payment to Union County:				
	Cash payment in addition to relocation work performed by Authority, for release of flood liability.				141,000.00
	Total account No. 204				2,149,076.30

204 Relocating highways and highway bridges—Detail by components:

Account Number	Land rights, highway easements	Land privilege, acquisition expense	Contract work	TVA force account	Release of liability	Total
204-01	$10,443.32	$12,510.72	$288,970.25	$163,770.48		$475,694.77
-02	2,150.00	695.04	115,509.49	472.58		118,827.11
-03	4,079.94	8,340.48	44,204.32	422.97		57,047.71
-04	199.50	1,042.56		15,711.40		16,953.46
-05			187,537.33	162,257.71		349,795.04
-100	1,259.50	2,432.64		83,125.31		86,817.45
-101	629.25	3,127.68		22,407.07		26,164.00
-102	2,793.46	11,468.15		152,712.82		166,974.43
-103	212.00	347.52		12,329.96		12,889.48
-104	617.00	1,390.08		22,880.31		24,887.39
-105	4,487.50	11,120.64		116,569.71		132,177.85
-106				19,696.08		19,696.08
-107	991.10	2,780.16		11,985.15		15,756.41
-108				18,216.69		18,216.69
-109	1,248.00	4,170.24		17,034.80		22,453.04
-110				2,948.93		2,948.93
-111				2,565.46		2,565.46
-112	591.00	695.04		52,005.47		53,291.51
-113	819.00	1,390.08		14,581.15		16,790.23
-114	2,143.75	5,560.32	32,205.24	2,454.51		42,363.82
-115	386.00	1,042.56		13,667.22		15,095.78
-116				34,800.50		34,800.50
-117			16,614.34	3,418.33		20,032.67
-118	827.75	3,475.20		24,947.37		29,250.32
-119	10.00	347.52		73,371.13		73,728.65
-120	20.00	347.52	4,122.02	.885.00		5,374.54
-121				1,268.74		1,268.74
-122				29,514.74		29,514.74
-123			24,820.17			24,820.17
-2	1,936.86	9,035.52	1,306.12	73,017.74		85,296.24
-3				26,583.09		26,583.09
-4					$141,000.00	141,000.00
	35,844.93	81,319.67	715,289.28	1,175,622.42	141,000.00	2,149,076.30

TABLE 100-C.—*Final project cost—details*—Continued

LAND AND LAND RIGHTS—Continued

Account	Description	Quantity	Unit	Rate	Amount
205	Relocating railways and railway bridges:				
-1	Southern Railway relocation—Relocating 2.8 miles Middlesboro Branch:				
	Grading				$266,084.04
	Tunnels				147,024.60
	Sycamore Creek Bridge				141,837.72
	Clinch River Bridge				208,917.21
	Six culverts, concrete				45,591.00
	Pipe culverts, various sizes				10,237.18
	Powell River Bridge Protection				10,541.03
	Track laying				51,598.91
	Shifting track				2,529.46
					884,361.15
-2	L. & N. Railroad relocation—Cove Creek Bridge No. 80:				
	Constructed four concrete piers, underpinned pedestals, and erected three deck plate girders as well as the reinforcing of two existing girders.				108,598.26
	Total account No. 205				992,959.41
207	Relocating other structures and improvements:				
-0	Relocating utilities—				
	Telephone lines:				
-01	Southern Bell Telephone Co				$7,241.00
	Individual subscriber lines				710.70
-02	Telegraph lines:				
	Western Union Telegraph Co				6,192.16
					$14,143.86
-1	Cemetery relocation (grave removal)	5,226	Grave	$15.39	80,433.98
-3	Relocating Families: This includes salaries and other expense involved in making a survey of all families living within the reservoir area, rendering assistance and planning their removal and readjustment in new locations; 2,847 families involved.				181,831.69
-7	Preservation of prehistoric material (archaeological).				4,161.34
	Total account No. 207				280,570.87
208	Protecting reservoir property:				
	Policing and forest fire prevention on reservation during construction.				78,873.57
	Total account No. 208				78,873.57
209	Protecting existing structures and improvements:				
-7	Caryville Dam				60,068.69
-8	Protection from seepage (McGhee tract)				7,148.71
	Total account No. 209				67,217.40
	Total land and land rights				12,168,826.24

STRUCTURES AND IMPROVEMENTS

Account	Description	Quantity	Unit	Rate	Amount
211	General preparation of site:				
-1	Remove rubbish, clear and grub				$3,966.68
	Total account No. 211				3,966.68
212	General yard improvements:				
-0	Grading and landscaping:				
-00	General grading and landscaping along dam approaches at overlook and at parking areas.				68,489.40
-02	Flag pole, memorial plaque, fountains, etc.				4,257.79

TABLE 100–C.—*Final project cost—details*—Continued

STRUCTURES AND IMPROVEMENTS—Continued

Account	Description	Quantity	Unit	Rate	Amount
211	General yard improvements—Con.				
-1	Roads, sidewalks, bridges, trestles:				
-11	Sidewalks, curbs and parapet walls at east end of dam.	939	Cubic yard	$33.51	$31,461.71
-12	Powerhouse approach road curb	45	Cubic yard	35.85	1,613.25
-2	Retaining walls, fences, gates and railings:				
-20	Retaining or property walls:				
-200	Concrete wall, southeast corner of powerhouse.	311	Cubic yard	13.86	4,311.39
-201	Concrete wall, east bank service road.				1,922.91
-202	Concrete wall, under headtower runway beam.	89	Cubic yard	24.78	2,205.61
-21	Fences protecting quarry				1,166.24
-4	Water filter and pumping system:				
-40	Drinking water system				12,316.35
-6	Sewers and drainage system				5,756.39
-9	Power and lighting (not part of any structure):				
-90	East approach				8,552.99
-91	West approach				2,401.12
-92	Flood lighting: Downstream face of dam.				1,803.99
	Total account No. 212				146,259.14
213	Powerhouse:				
-1	Diversion and care of water:				
-10	Protection from floods after abandonment of cofferdam No. 3.				26,852.54
-11	Cofferdam No. 4				36,280.49
-2	Excavation and backfill:				
-22	Earth excavation—general	61,623	Cubic yard	0.51	31,233.01
-23	Rock excavation—general	44,754	Cubic yard	2.28	102,222.76
-3	Foundation preparation and treatment:				
-30	Drilling grout holes				6,833.28
-4	Concrete:				
-40	Substructure concrete:				
-400	Concreting	21,368	Cubic yard	7.15	152,715.28
-401	Forms	77,043	Square foot	1.75	135,083.05
-402	Reinforcing steel	873,538	Pound	.039	34,288.59
	Total substructure concrete	21,368	Cubic yard	15.07	322,086.92
-41	Superstructure concrete:				
-410	Concreting	4,049	Cubic yard	16.23	65,695.31
-411	Forms	132,698	Square foot	1.12	148,855.00
-412	Reinforcing steel	456,694	Pound	.055	25,082.31
	Total superstructure concrete.	4,049	Cubic yard	59.18	239,632.62
-6	Superstructure:				
-61	Interior masonry:				
-610	Hollow tile partitions, 2 inch to 8 inch.	1,523	Cubic foot	1.27	1,934.05
-616	Acoustical ceiling	82	Square yard	3.52	288.70
-62	Steel and iron:				
-620	Structural steel	1,858,319	Pound	.053	99,114.41
-621	Miscellaneous steel and iron	96,840	Pound	.16	15,823.81
-624	Aluminum work				11,873.65
-625	Wire and steel partitions	4,465	Pound	.32	1,414.53
-626	Steel baseboard	304	Linear foot	3.09	939.23
-632	Kalamein and hollow metalwork:				
-6322	Hollow metal doors	1,985	Square foot	14.00	27,786.37
-6324	Window frames and sash	3,625	Square foot	4.92	17,828.62
-65	Marble and terrazzo, etc.:				
-651	Quarry tile	14,707	Square foot	.63	9,303.90
-652	Terrazzo work				2,864.82
-653	Linoleum	240	Square yard	2.61	626.56
-654	Burlap wall finish	74	Square yard	2.71	200.39
-655	Structural glass	2,259	Square foot	2.59	5,858.23
-66	Roofing and sheet metalwork:				
-660	Roofing and flashing:				
	Concrete slab, 5-ply roofing, concrete slab on pedestals, and flashing.	19,255	Square foot	1.31	25,299.02
-67	Plastering	1,171	Square yard	6.25	7,318.11
-68	Painting and glazing:				
-680	Painting				9,660.18
-681	Glass and glazing	3,744	Square foot	.41	1,544.98

TABLE 100-C.—*Final project cost—details*—Continued

STRUCTURES AND IMPROVEMENTS—Continued

Account	Description	Quantity	Unit	Rate	Amount
213	Powerhouse—Continued.				
-8	Service work:				
-80	Plumbing				$12, 542. 41
-81	Floor and roof drains				17, 854. 97
-82	Heating:				
-825	Fixtures: 32 assorted electric space heaters.				4, 333. 11
-83	Air conditioning				15, 258. 75
-84	Ventilating system				6, 450. 12
-89	Lighting:				
-895	Fixtures, switches, receptacles				11, 560. 26
	Total account No. 213				1, 072, 820. 80
217	Air, water, and automotive terminals:				
-1	Automobile parking area				13, 899. 60
	Total account No. 217				13, 899. 60
	Total structures and improvements				1, 236, 946. 22

RESERVOIRS, DAMS, AND WATERWAYS

Account	Description	Quantity	Unit	Rate	Amount
220	Reservoir:				
-0	Clearing	18, 693	Acre	$61. 00	$1, 140, 358. 32
-6	Rim treatment:				
-60	Drilling grout holes 2½-inch and 3-inch core drill and 6-inch well drill holes.				138, 018. 13
-61	Grouting	257, 736	Cubic foot	. 54	138, 091. 27
-62	Excavation	402	Cubic yard	19. 00	7, 638. 28
-63	Concrete	402	Cubic yard	9. 55	3, 840. 70
	Total account No. 220				1, 427. 946. 70
221	Concrete dam and spillway:				
-0	Exploration of foundation:				
-00	Test pits: Excavation, grouting, concreting.				19, 083. 37
-02	Drilling and geophysical investigation.				10, 256. 43
-1	Diversion and care of water:				
-11	Coffer damNo. 1				172, 847. 53
-12	Cofferdam No. 2				46, 965. 29
-13	Cofferdam No. 3, including bridge and ramp.				119, 537. 59
-15	Cofferdam No. 5				5, 042. 54
-19	Final closure				13, 734. 26
-2	Excavation and backfill:				
-22	Earth excavation, general	124, 872	Cubic yard	. 51	63, 290. 13
-23	Rock excavation, general	241, 261	Cubic yard	2. 28	551, 002 72
-24	Backfilling	17, 147	Cubic yard	. 91	15, 584. 15
-3	Foundation preparation and treatment:				
-30	Drilling grout holes, 2½-inch wagon drill and 3-inch and 5½-inch shot core drill holes.				181, 969. 84
-31	Pipe and fittings				10, 396. 39
-34	Seam washing				43, 005. 54
-35	Seam tunnels:				
-350	Excavation	2, 940	Cubic yard	9. 69	28, 483. 21
-351	Concrete	2, 531	Cubic yard	6. 50	16, 458. 02
-37	Pressure grouting	203, 054	Cubic foot	1. 26	255, 833. 17
-38	Foundation drains (metal, concrete, wood, etc.).				7, 878. 42
-4	Concrete:				
-40	Mass concrete:				
-400	Concreting	872, 109	Cubic yard	4. 53	3, 952, 749. 22
-401	Forms	1, 132, 550	Square foot	. 53	603, 131. 55
-402	Reinforcing steel	2, 177, 465	Pound	. 039	84, 260. 62
	Total mass concrete	872, 109	Cubic yard	5. 32	4, 640, 141. 39
-41	Spillway crest and piers:				
-410	Concreting	13, 248	Cubic yard	6. 31	83, 561. 79
-411	Forms	84, 449	Square foot	. 94	79, 799. 58
-412	Reinforcing steel	457, 274	Pound	. 049	22, 593. 61
	Total spillway crest and piers concrete.	13, 248	Cubic yard	14. 04	185, 954. 98

Table 100–C.—*Final project cost—details*—Continued

RESERVOIRS, DAMS, AND WATERWAYS—Continued

Account	Description	Quantity	Unit	Rate	Amount
221	Concrete dam and spillway—Continued.				
	Concrete—Continued.				
-42	Spillway apron:				
-420	Concreting	26,161	Cubic yard	$5.60	$146,612.89
-421	Forms	37,620	Square foot	.96	36,041.11
-422	Reinforcing steel	163,547	Pound	.034	5,614.85
	Total spillway apron concrete.	26,161	Cubic yard	7.20	188,268.85
-43	Reinforced training walls:				
-430	Concreting	8,068	Cubic yard	6.35	51,215.73
-431	Forms	37,874	Square foot	.83	31,491.23
-432	Reinforcing steel	739,099	Pound	.035	25,766.41
	Total reinforced training wall concrete.	8,068	Cubic yard	13.44	108,473.37
-44	Gravity training wall:				
-440	Concreting	11,558	Cubic yard	4.79	55,382.31
-441	Forms	34,294	Square foot	.68	23,344.93
	Total gravity training wall concrete.	11,558	Cubic yard	6.81	78,727.24
-45	Outlet trashrack structures:				
-450	Concreting	3,639	Cubic yard	6.15	22,375.83
-451	Forms	40,340	Square foot	.90	36,433.76
-452	Reinforcing steel	198,136	Pound	.043	8,472.29
	Total outlet trashrack structure concrete.	3,639	Cubic yard	18.49	67,281.88
-47	Intake trashrack structures:				
-470	Concreting	1,503	Cubic yard	11.55	17,366.64
-471	Forms	37,694	Square foot	1.46	55,078.81
-472	Reinforcing steel	202,854	Pound	.044	8,955.72
	Total intake trashrack structure concrete.	1,503	Cubic yard	54.16	81,401.17
-5	Joints, stops, waterproofing, and drains:				
-54	Grout pipe and boxes for pressure grouting.				38,356.28
-56	Copper, water, and grout stops	42,985	Pound	.46	19,987.01
-57	Core drilling for contraction grouting.	1,224	Linear foot	3.66	4,484.36
-58	Drains (formed or porous tile, etc.)	23,593	Linear foot	1.74	41,006.33
-59	Gallery drainage (pumps and piping).	1	System		4,397.56
-6	Gates and appurtenances:				
-60	Outlet gates (eight sluiceways):				
-601	Gates and conduit liners	2,030,490	Pound	.073	148,312.59
-602	Operating equipment				62,444.38
-603	Trashracks and supports	525,033	Pound	.033	17,185.54
-61	Crest gates:				
-611	Three drum gates	1,279,412	Pound	.10	132,356.23
-612	Operating equipment				32,142.33
-62	Intake gates:				
-620	Two tractor gates, operating equipment, guides, seats, and emergency gate guides.				212,037.14
-623	Trashracks and supports	765,824	Pound	.040	30,805.10
-624	Trash cleaning system				6,205.03
-7	Auxiliary structures or equipment:				
-71	Elevator				15,291.18
-72	Bridges, walkways, culverts				1,752.38
-74	Miscellaneous iron:				
-740	Floor plates, grating, and doors				3,266.26
-741	Ladders and stairs	16,058	Pound	.35	5,662.88
-75	Handrailing				1,710.44
-76	Gallery ventilation				94.43
-79	Elevator penthouse				12,215.43
-9	Electric work:				
-93	Protective equipment:				
-932	Equipment grounds	4,767	Pound	.97	4,613.29
-95	Conduit work:				
-950	Concealed steel	92,967	Pound	.18	16,770.79
-951	Exposed steel	40,577	Pound	.26	10,606.73
-953	Nonmetallic	4,983	Pound	.17	852.30
-954	Concrete envelopes	20	Cubic yard	23.13	462.63
-955	Boxes and cabinets				4,590.50
-956	Concrete vault, manhole frame and cover.				1,196.05

TABLE 100-C.—*Final project cost—details*—Continued

RESERVOIRS, DAMS, AND WATERWAYS—Continued

Account	Description	Quantity	Unit	Rate	Amount
221	Concrete dam and spillway—Continued.				
	Electric work—Continued.				
-96	Power wiring:				
-960	Braided cable and wire				$4,059.65
-962	Armored cable				3,756.74
-97	Control wiring:				
-970	Braided cable and wire				1,247.08
-971	Leaded cable				372.05
-972	Armored cable				1,524.04
-980	Lighting transformers	75	Kilovolt amp	$5.83	437.03
-993	Lighting fixtures	304	Outlet	10.85	3,299.35
	Total account No. 221				7,755,118.59
223	East embankment:				
-2	Excavation and backfill:				
-21	Stripping				1,892.53
-27	Core trench excavation, including 331,128 feet board measure of shoring.	6,021	Cubic yard	17.29	104,108.43
-3	Foundation preparation and treatment:				
-30	Drilling grout holes	17,246	Linear foot	2.64	45,483.63
-35	Seam tunnels:				
-350	Excavation	2,248	Cubic yard	18.91	42,502.84
-351	Concrete	2,479	Cubic yarde	10.12	25,090.29
-4	Cut-off wall (core wall):				
-43	Concrete wall:				
-430	Concreting	5,988	Cubic yard	7.59	45,471.91
-431	Forms	12,902	Square foot	.79	10,246.24
-432	Reinforcing steel	376,759	Pound	.033	12,431.34
	Total cut-off wall concrete	5,988	Cubic yard	11.38	68,149.49
-7	Rolled fill:				
-72	Spreading and rolling	71,608	Cubic yard	.77	55,283.55
-8	Slope protection:				
-81	Riprap	4,173	Cubic yard	7.05	29,405.94
-83	Seeding and sodding				2,462.73
-84	Drainage				6,004.35
	Total account No. 223				380,383.78
224	Buffalo Creek divide dike (Loyston dike):				
-2	Excavation and backfill				1,527.28
-3	Foundation preparation and treatment.				15,427.08
-7	Rolled fill	80,458	Cubic yard	.64	51,626.52
-8	Slope protection:				
-81	Riprap	7,700	Cubic yard	.92	7,119.42
-83	Top soil, seeding, sodding				4,676.85
	Total account No. 224				80,377.15
226	Water conductors:				
-3	Penstock pipe	1,241,534	Pound	.10	125,008.74
-8	Tailrace:				
-80	Paving, concrete	117	Cubic yard	6.97	815.80
-83	Riprap, including concrete toe				28,762.34
	Total account No. 226				154,586.88
	Total reservoirs, dams, and waterways.				9,798,413.10

WATER WHEELS, TURBINES, AND GENERATORS

Account	Description	Quantity	Unit	Rate	Amount
230	Foundations and miscellaneous steel and iron:				
-3	Draft tube pier nose castings	61,930	Pound	$0.058	$3,586.80
	Total account No. 230				3,586.80
231	Turbines, including scroll cases, speed rings, and draft tube liners:				
	(Francis type 66,000 horsepower)	2	Turbine	293,449.64	586,899.27
	Total account No. 231				586,899.27

TABLE 100-C.—*Final project cost—details*—Continued

WATER WHEELS, TURBINES, AND GENERATORS—Continued

Account	Description	Quantity	Unit	Rate	Amount
232	Auxiliary equipment for turbines:				
-0	Governors	2	Governor	$17,737.57	$35,475.14
-91	Piezometer piping				4,039.14
	Total account No. 232				39,514.28
235	Generators—2 at 56,000 kilovolt-amp	100,800	Kilowatt	9.26	933,764.71
	Total account No. 235				933,764.71
236	Auxiliary equipment for generators:				
-0	Excitation system	2	System	1,352.41	2,704.81
-1	Generator cooling, heating, and ventilating system.	2	System	2,519.87	5,039.74
	Total account No. 236				7,744.55
239	Miscellaneous equipment for turbines and generators:				
-0	Generator and turbine lubrication and turbine cooling.	2	System	1,549.34	3,098.68
-3	Hydraulic gage board	2	Board	3,115.63	6,231.25
	Total account No. 239				9,329.93
	Total water wheels, turbines, and generators.				1,580,839.54

ACCESSORY ELECTRIC EQUIPMENT

Account	Description	Quantity	Unit	Rate	Amount
241	Switchgear:				
-0	Assembled switchgear:				
-00	Potential and surge protection cubicle.	2	Cubicle	$3,954.35	$7,908.69
-01	Neutral O. C. B and cubicle	2	O. C. B	1,563.39	3,126.78
-2	Disconnecting switches: 2 manual, 2 motor operating, 15-kilovolts.	4	Switch	557.86	2,231.42
-3	Instrument transformers: 8 13,800-volts potential and 34 15,000-volts current transformers.	42	Transformer	156.64	6,578.92
	Total account No. 241				19,845.81
242	Switchboards:				
-0	Main control boards: Back to back instrument and relay board, and benchboard.	1	Board		38,390.02
-1	Control terminal cabinets for 22 switchboard panels.				17,876.51
-3	440-volts alternating-current boards	5	Board		26,443.98
-5	Battery boards	1	Board		2,931.20
-8	Load and frequency control and standard frequency source.	1			14,017.73
-9	Annunciator equipment				10,611.10
	Total account No. 242				110,270.54
243	Protective equipment:				
-2	Grounding system	10,483	Pound	0.62	6,450.27
-4	Neutral reactors and cubicles	2	Reactor	1,579.29	3,158.57
	Total account No. 243				9,608.84
244	Electrical structures:				
-1	Metal enclosures (bus housings)	2	Enclosure	3,283.75	6,567.49
-4	Cable trays and racks: 15,057 pounds galvanized steel, 4,745 pounds transite.				3,150.58
	Total account No. 244				9,718.07
245	Conduit work:				
-0	Concealed steel	69,525	Pound	.32	22,590.85
-1	Exposed steel	25,158	Pound	.35	8,896.34
-3	Nonmetallic: 4-inch transite	11,109	Pound	.13	1,437.21
-5	Pull boxes and cabinets				4,132.79
	Total account No. 245				37,057.19

TABLE 100–C.—*Final project cost—details*—Continued

ACCESSORY ELECTRIC EQUIPMENT—Continued

Account	Description	Quantity	Unit	Rate	Amount
246	Power wiring:				
-0	Braided cable:				
-00	Low voltage				$9, 598. 25
-01	High voltage				1, 070. 08
-4	Bare bars, tubes, shapes	2,977	Pound	$0. 63	1, 875. 25
-5	Insulators and bushings, 15-kilovolt				430. 25
-7	Power receptacles (except lighting)	20	Outlet	26. 44	528. 89
	Total account No. 246				13, 502. 72
247	Control wiring:				
-0	Braided wire and cable				8, 130. 25
	Total account No. 247				8, 130. 25
249	Station service equipment:				
-0	Transformers:				
-00	Auxiliary power, two 750-kilovolt-ampere, self-cooled, 14,200/460-volt, 3-phase.	1, 500	Kilovolt-ampere.	4. 68	7, 023. 33
-01	Lighting and heating, 100- and 75-kilovolt-ampere, 480–240/120-volt, single-phase.	175	Kilovolt-ampere.	5. 53	967. 13
-1	Control battery and charging equipment.				7, 332. 87
-8	Transil and lubricating oil storage, purifying, and piping system.	1	System		16, 353. 56
	Total account No. 249				31, 676. 89
	Total accessory electric equipment				239, 810. 31

MISCELLANEOUS POWER PLANT EQUIPMENT

Account	Description	Quantity	Unit	Rate	Amount
252	Station maintenance equipment:				
-0	Machine shop equipment				$12, 797. 04
	Total account No. 252				12, 797. 04
255	Cranes and hoisting equipment:				
-0	250-ton generator room crane:				
-00	Crane equipment				72, 557. 65
-01	Crane rails and supports	26, 400	Pound	$0. 059	1, 549. 37
-02	Contact rails and supports	186. 5	Linear foot	9. 77	1, 822. 72
	Total account No. 255				75, 929. 74
256	Compressed air and vacuum cleaning systems:				
-0	Compressed air system:				
-00	Compressors and air receivers: (1 at 330; 1 at 45; 1 at 8 cubic feet a minute.)	383	Cubic foot a minute.	18. 08	6, 923. 10
-01	Piping				1, 628. 94
	Total account No. 256				8, 552. 04
258	Station service water system:				
-1	Pipe, fittings, meters				12, 183. 86
	Total account No. 258				12, 183. 86
259	Other miscellaneous equipment:				
-0	Telephone system:				
-00	Cable and wire				5, 236. 55
-01	Equipment				12, 528. 75
-1	Fire extinguishing equipment:				
-10	Portable fire extinguishers (30 assorted).				1, 216. 72
-11	Carbon dioxide system				12, 433. 62
-2	Office furniture and equipment:				
-20	Powerhouse				3, 279. 78
-21	Reception room				3, 062. 26
-4	Portable meters and instruments (oscillograph and megger).				2, 928. 02
-5	Gauges and Indicators:				
-50	Electric clocks				729. 69
-51	Water level indicators				2, 342. 71

TABLE 100-C.—*Final project cost—details*—Continued

MISCELLANEOUS POWER PLANT EQUIPMENT—Continued

Account	Description	Quantity	Unit	Rate	Amount
259	Office furniture and equipment—Contd.				
-6	Carrier current relay and intersite communication.				$31,608.16
	Total account No. 259				75,366.26
	Total miscellaneous power plant equipment.				184,828.94

ROADS, RAILROADS, AND BRIDGES

Account	Description	Quantity	Unit	Rate	Amount
260	Access roads for permanent use:				
-2	Approach roads:				
-20	East dam approach—Dam to 400 feet east of dam, traffic circle, and powerhouse road.				$31,343.46
-24	West dam approach—Concrete structure.	1,477	Cubic yard	$12.07	17,824.74
	Total account No. 260				49,168.20
262	Roadway on concrete dam:				
-4	Concrete:				
-41	Spillway bridge:				
-410	Concreting	451	Cubic yard	15.57	7,021.86
-411	Forms	17,436	Square foot	0.88	15,309.34
-412	Reinforcing steel	78,067	Pound	0.045	3,519.70
	Total spillway bridge concrete	451	Cubic yard	57.32	25,850.90
-42	Parapet walls and sidewalks:				
-420	Concreting	774	Cubic yard	23.79	18,411.35
-421	Forms	20,627	Square foot	1.47	30,239.25
-422	Reinforcing steel	60,918	Pound	0.053	3,233.61
	Total parapet wall and sidewalk concrete	774	Cubic yard	67.03	51,884.21
-43	Parapet walls and sidewalks on core wall (concrete)	330	Cubic yard	25.04	8,262.01
-5	Permanent or wearing surface:				
-50	Roadway surfacing	4,019	Square yard	2.20	8,859.56
-51	Drainage				6,495.85
-6	Structural steel	772,199	Pound	0.051	39,439.86
-9	Lighting:				
-92	Fixtures and wiring				18,145.41
	Total account No. 262				158,937.80
	Total roads, railroads and bridges				208,106.00

TABLE 100-C.—*Final project cost—details*—Continued

DIRECT COST OF TRANSMISSION PLANT

STRUCTURES AND IMPROVEMENTS

Account	Description	Quantity	Unit	Rate	Amount
421	General preparation of site:				
	Excavation including foundation investigation and fine grading.	16,008	Cubic yard	$0.77	$12,322.87
	Total Account No. 421				12,322.87
422	General yard improvements:				
-0	Grading and landscaping				2,405.10
-21	Fences, gates, and railings				2,671.83
-6	General drainage				2,402.70
	Total account No. 422				7,479.63
	Total structures and improvements				19,802.50

STATION EQUIPMENT

Account	Description	Quantity	Unit	Rate	Amount
430	Outdoor substation structure:				
-1	Foundations for structures and equipment.				$36,352.08
-3	Structural steel	275,372	Pound	$0.091	24,976.86
-7	Cable and pipe tunnel:				
-70	Excavation and backfilling	607	Cubic yard	22.34	13,559.54
-74	Concrete	1,202	Cubic yard	24.67	29,652.03
-75	Miscellaneous iron, covers, etc				1,352.30
-76	Cable trays and supports				3,388.94
-9	Lighting:				
-90	Transformers (2 at 10 kilovolt-amperes).		Kilovolt-ampere	11.81	236.11
-91	Cabinets				188.80
-92	Conduit	3,130	Pound	.49	1,546.94
-93	Wiring				956.88
-94	Fixtures				1,244.35
	Total account No. 430				113,454.83
431	Switchgear:				
-1	Oil circuit breaker	6	Breaker	31,028.38	186,170.30
-2	Disconnecting switch	15	Switch	2,069.26	31,038.85
	Total account No. 431				217,209.13
433	Protective equipment:				
-0	Arrestors	2	Arrestor	4,542.92	9,085.83
-2	Grounding system	17,841	Pound	.59	10,612.16
-4	Impedors (2 at 12,600 kilovolt-amperes)	25,200	Kilovolt-ampere	.35	8,853.26
	Total account No. 433				28,551.25
435	Conduit work:				
-0	Concealed steel	13,894	Pound	.27	3,692.26
-1	Exposed steel	3,245	Pound	.42	1,372.91
-3	Nonmetallic 4-inch transite	22,714	Pound	.15	3,357.67
-5	Boxes				330.07
	Total account No. 435				8,752.91
436	Power wiring:				
-1	Leaded cable:				
-10	Lighting and battery				2,308.78
-11	Main power	7,826	Linear foot	2.82	22,038.08
-3	Bare cable	4,763	Pound	.88	4,193.08
-4	Bare bars, tubes, shapes	14,493	Pound	.43	6,187.49
-5	Insulators:				
-50	Pedestal				7,061.92
-51	Strain				3,731.71
	Total account No. 436				45,521.06

TABLE 100-C.—*Final project cost—details*—Continued

STATION EQUIPMENT—Continued

Account	Description	Quantity	Unit	Rate	Amount
437	Control wiring:				
-1	Leaded cable				$9,288.85
	Total account No. 437				9,288.85
438	Main conversion equipment:				
-0	Power transformers:				
	6 at 18,667 kilovolt-amperes, 13,800-110,000/154,000 volt.	112,000	Kilovolt-ampere	$2.56	286,821.90
	Total account No. 438				286,821.90
439	Station service equipment:				
-4	Transformer handling equipment:				
-40	Tracks	39,187	Pound	.082	3,210.39
-41	Transfer car and electric lift	1			7,037.02
-6	Transil oil distribution system:				
-60	Piping				6,194.51
-62	Oil in storage	6,333	Gallon	.31	1,994.54
-95	Fire protection equipment				721.71
	Total account No. 439				19,158.17
	Total station equipment				728,758.10

DIRECT COST OF GENERAL PLANT

Account	Description	Quantity	Unit	Rate	Amount
	NORRIS				
	Land costs:				
	Norris Freeway from highway Tennessee 33 to West Norris Road.	148.6	Acre	$87.02	$12,931.36
	Direct construction costs:				
	Norris Freeway from highway Tennessee 33 to West Norris Road. Highway construction including $231,024.68 contract work.	13.0	Mile	46,972.49	610,642.32
	OTHER				
	Land costs:				
	Norris Freeway from highway U. S. 25W to Coal Creek Yard.	6.9	Acre	202.45	1,396.88
	Direct construction costs:				
	Norris Freeway from highway U. S. 25W to Coal Creek Yard. Highway construction; all contract work.	.26	Mile	47,936.81	12,463.57

TABLE 100-C.—*Final project cost—details*—Continued

INDIRECT CONSTRUCTION COSTS

Other general costs—Norris freeway distribution

Account	Description	Quantity	Unit	Rate	Amount
	Land costs:				
	Highway U. S. 25W to Norris Dam construction office.	116.78	Acre	$73.80	$8,618.59
					8,618.59
	Direct construction costs:				
	Highway U. S. 25W to Norris Dam construction office.	4.05	Mile	61,195.36	247,841.21
	West approach road: Dam office to 126 feet west of dam; highway construction.	.99	Mile	78,173.07	77,391.34
	East approach road: 400 feet east of dam to powerhouse intersection; highway construction.	.89	Mile	20,937.17	18,634.08
	Powerhouse intersection to West Norris Road; highway construction.	2.50	Mile	67,494.08	168,735.19
					512,601.82

Camp and other indirect costs

Superintendence, accounting, and timekeeping: Salaries and expenses in connection with superintendent of construction and his immediate assistants in the field and office, also salaries and expenses in connection with field accounting and timekeeping offices	$283,313.86
Transportation: Personal transportation and other expense incurred by field superintendents, etc., in maintaining contact with working units	87,838.41
Office supplies and expense: Miscellaneous expense of field office, such as stationery and office supplies, blue prints, photostats, telephone and telegraph, maintenance of office space, etc	136,230.19
Construction plant expense: Planning, design, and expense of the construction plant and equipment, together with certain unallocable items of cost, such as maintaining county roads	73,251.03
Camp operation: Net expense of operation of Norris Town and camp, including depreciation and training activities during construction period. (See p. 219)	918,999.50
Provision for medical service: Provision for costs of medical examinations and treatment for service-connected injuries or disabilities. (Allocation based on pay roll distribution)	176,029.59
Malaria prevention: Artificial control of mosquitoes	8,155.04
Police and guide service and accommodation of guests: Police and guide service maintained for benefit of project and accommodation of guests when on official visits	63,923.31
Other credits: Liquidated and miscellaneous damages, less bonus payments and other costs incurred in cancellation of contracts due to change in design or policy	Cr. 6,150.52
Total camp and other indirect costs	[1] 1,741,590.41

[1] Expanded by activities shown in Table 101.

TABLE 100-C.—*Final project cost—details*—Continued

DISTRIBUTIVE GENERAL EXPENSE

Design and construction engineering costs

Engineering administration: Provision for salary and expenses of chief engineer's general administrative office. (Allocation based on pay-roll distribution during fiscal years 1937 and 1938 and in prior years on estimates)	$50, 958. 75
Engineering—field and office: Salaries and expenses of executive and supervisory engineers, cost engineers and assistants, concrete technicians, inspectors and assistants, shop tests and inspection of miscellaneous materials not assigned to specific items of property, also engineering for control lines, bench marks, etc	1, 263, 401. 14
Dam site and regional geology: Geologic examinations and studies made of the dam foundation and reservoir rim	28, 000. 51
Design: Design of dam, powerhouse, switchyard and other related structures, including $270,845.75 for U. S. Bureau of Reclamation charges, architecture of dam and powerhouse and landscape treatment of dam site	482, 349. 93
Consulting service: Salary and expenses including travel and subsistence of consultants	2, 784. 07
Total design and construction engineering costs	[1] 1, 827, 494. 40

Executive and administrative costs

General administrative expense: Provision for costs of general office, including board of directors, general manager's office, finance department, legal department (exclusive of land condemnation costs), materials department, personnel department, and office service department. (Allocation based on pay roll distribution)	1, 001, 804. 41
Division administrative costs: Provision for costs of general office expense of the various divisions, such as construction and maintenance, reservoir clearing, highway and railroad, cemetery relocation, etc. (Allocation based on pay roll distribution.)	55, 617. 10
Total executive and administrative costs	[1] 1, 057, 421. 51
Maps and Surveys: Establishment of basic horizontal and vertical positions around the reservoir, surveys and mapping of the topography at the dam site and the Loyston divide area, aerial photography and mosaic construction, surveys to provide the basic data for studies of silt deposits in the reservoir, and other general surveys and mapping	134, 866. 24
Hydraulic data: Prorata share of the cost of collection and compilation of basic hydrographic and hydrologic data, such as river forecasting, rainfall and evaporation studies, measurements of the amounts of silt carried by various streams and deposited in reservoirs, stream flow measurements, ground water investigations, laboratory tests of hydraulic structures, etc	93, 160. 38
Project planning: Prorata share of the cost of investigations and studies of projects proposed or under construction, such as studies made to determine the feasibility of the project, preliminary plans and estimates, studies of stream flow and flood data, determination of most economical power installations, navigation studies, etc	22, 652. 91
Regional planning studies: Miscellaneous preliminary investigations, such as a report on lands served by the LaFollette branch of the Southern Railway and plans for use of land in Norris Reservoir area	21, 268. 36

[1] Expanded by activities in Table 101.

TABLE 100-C.—*Final project cost—details*—Continued

Executive and administrative costs—Continued

Final project report: Cost of assembling and editing a report on the planning, design, construction, and initial operations of the project	$9,620.19
Property records: Analysis of actual costs of each inventoriable item, the recording of same in permanent property records, and the preparation of the final report on the cost of the project	35,967.24
Total other general costs	317,535.32

TABLE 101.—*Details of certain indirect construction costs and distributive general expenses by activities*

	Relocations and protecting structures [1]	Reservoir	Dam and powerhouse construction	Norris Freeway	Total
INDIRECT CONSTRUCTION COSTS					
Camp and other indirect costs:					
Superintendence	$9,428.55	$52,831.88	$50,348.95	$2,745.55	$283,313.86
Accounting and timekeeping	41,060.71		126,033.01	865.21	
Transportation	28,296.59	23,146.87	35,963.78	431.17	87,838.41
Office supplies and expense	20,848.15	6,417.17	99,119.16	9,845.71	136,230.19
Construction plant expense			38,624.89	34,626.14	73,251.03
Camp operation			[2] 918,999.50		918,999.50
Provision for medical service	16,468.97	8,017.58	144,788.04	6,755.00	176,029.59
Malaria prevention			8,155.04		8,155.04
Police and guide service, accommodation of guests, and safety activities			63,923.31		63,923.31
Public liability	1,018.10			402.50	[3] 6,150.52
Other credits less bonus payments	[3] 3,314.25		[3] 5,895.27	1,638.40	
Total	113,806.82	90,413.50	1,480,060.41	57,309.68	1,741,590.41
DISTRIBUTIVE GENERAL EXPENSE					
Design and construction engineering costs:					
Engineering administration	2,340.65		48,618.10		50,958.75
Engineering—field and office	513,797.33	58,829.60	575,874.22	114,899.99	1,263,401.14
Dam site and regional geology			28,000.51		28,000.51
Design by Authority			211,504.18		482,349.93
Design by Bureau of Reclamation			270,845.75		
Consulting services			2,784.07		2,784.07
Total	516,137.98	58,829.60	1,137,626.83	114,899.99	1,827,494.40
Executive and administrative costs:					
General administrative costs	106,908.47	136,263.11	737,279.19	21,353.64	1,001,804.41
Division administrative costs	29,350.24	24,294.35		1,972.51	55,617.10
Total	136,258.71	160,557.46	737,279.19	23,326.15	1,057,421.51

[1] Relocating highways, railroads, and other structures and improvements; protecting existing structures and improvements.
[2] See p. 219 for details of camp operation.
[3] Denotes credit.

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*TVA staff member

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Appendix A

STATISTICAL SUMMARY [1]

Authorized by	Tennessee Valley Authority Act, 73d Cong., May 18, 1933
Work started	Oct. 1, 1933
Placed in operation:	
Flood control	Mar. 16, 1936
Water releases for navigation	June 19, 1936
Power operations	July 28, 1936

Location

On Clinch River in Campbell and Anderson Counties, Tenn.:	
Above mouth	79.8 miles
From Norris, Tenn.	6 miles
Below confluence of Powell and Clinch Rivers	8.8 river miles
Above Clinton, Tenn.	20 river miles
North-northwest of Knoxville, Tenn.	20 air miles
East-northeast of Wilson Dam, Ala.	225 air miles

Stream Flow

Drainage area at dam	2,912 square miles
Gaging station records:	
Clinton, October 1903 to May 1927, drainage area	3,056 square miles
Coal Creek, May 1927 to date, drainage area	2,921 square miles
Below Norris Dam, June 1936 to date, drainage area	2,913 square miles
Maximum flow (1886), estimated	115,000 cubic feet per second
Average flow	4,600 cubic feet per second
Minimum flow (1909, 1910, 1911)	200 cubic feet per second

Reservoir and Water Elevations

Counties affected:	
State of Tennessee	Anderson, Campbell, Claiborne, Grainger, Union
Operating levels at dam:	
Maximum assumed for structural design (area 49,500 acres, approximate)	elevation 1,052
Probable maximum high water (46,680 acres, approximate)	elevation 1,047
Top of gates (area 40,160 acres, approximate)	elevation 1,034
Spillway crest (area 34,200 acres, approximate)	elevation 1,020
Minimum expected (area 13,500 acres, approximate)	elevation 955
Length at elevation 1,020 (backwater):	
Clinch	72 river miles
Powell, above junction	56 river miles
Length at elevation 1,020 (backwater):	
Clearing (18,693 acres):	
Below elevation 940 (see p. 513)	4,961 acres
Between elevation 940–1,020	13,732 acres
Original river area (below elevation 1,020)	2,913 acres
Storage (flat pool assumption):	
Uncontrolled flood storage (elevation 1,034–1,052)	803,000 acre-feet
Total volume at elevation 1,034	2,567,000 acre-feet
Total volume at elevation 1,020	2,047,000 acre-feet
Total controlled volume between minimum possible draw-down, elevation 860, and top of gates, elevation 1,034	2,550,000 acre-feet
Total volume below minimum possible draw-down, elevation 860	17,000 acre-feet

[1] All elevations are based on 1912 Fourth General Adjustment; to correct to 1936 Supplemental Adjustment add 0.12 foot. Contractors for major equipment items and the cost of these items are listed in appendix G, pp. 784 to 786.

Reservoir and Water Elevations—Continued

TAIL WATER

Minimum ---------- elevation 820.5
Average ---------- elevation 826.0
Maximum recorded level (1886), estimated ---------- elevation 860.0

HEAD (GROSS)

Average maximum ---------- 194 feet
Average minimum ---------- 129 feet
Maximum possible:
One unit floating on the line, H.W. 1,034, T.W. 820.5 ---------- 213.5 feet
Maximum probable:
Both units running at capacity, H.W. 1,034, T.W. 827 ---------- 207 feet

Navigation Facilities

Increase in minimum dry-season flows downstream at completion of Norris Dam (approximate) ---------- 6,000 cubic feet per second
Corresponding increase in controlling navigable depths at low water:
Above Chattanooga ---------- 1.7 feet
Between Pickwick Landing and mouth ---------- 1.5 feet
On lower Mississippi River at Memphis ---------- 0.6 feet
Available 9-foot navigation on reservoir:
Clinch River:
At elevation 1,020 ---------- 71 river miles
At elevation 955 ---------- 53 river miles
Powell River:
At elevation 1,020 ---------- 52 river miles
At elevation 955 ---------- 35 river miles
Cove Creek, Big Creek, Cedar Creek, and Davis Creek:
Total at elevation 1,020 ---------- 33 river miles
Total at elevation 955 ---------- 20 river miles

Dam

Type and material ---------- Nonoverflow concrete gravity sections; concrete gravity spillway section, gate controlled; earth fill section with concrete core wall.
Length:
Gravity section ---------- 1,570 feet
Earth fill section ---------- 290 feet

Total ---------- 1,860 feet
Maximum height of dam, foundation to roadway (elevation 1,061) ---------- 265 feet
Maximum width at base:
Spillway section only ---------- 208 feet
Including apron ---------- 423 feet
Length of concrete core wall ---------- 584 feet
Concrete in dam (including powerhouse and other permanent structures) ---------- 1,002,253 cubic yards
Earth in east embankment ---------- 71,608 cubic yards
Roadway width, crown at elevation 1,061 ---------- 22 feet
Foundation ---------- Rock (Knox dolomite)
Toe protection ---------- Pool with concrete floor
Top of embankments ---------- elevation 1,061
Design ---------- Gravity action; downstream slope 0.70
Uplift: reservoir pressure upstream and tail water pressure downstream, acting over ⅔ area of base; maximum sliding factor 0.65; concrete weight, 150 pounds per cubic foot.

SPILLWAY SECTION

Type ---------- Overflow and outlet conduits with hydraulic jump stilling pool
Length ---------- 300 feet clear; 332 feet total. Crest at elevation 1,020
Capacity:
Overflow with gates down, water elevation 1,034 ---------- 54,000 cubic feet per second

DAM—Continued

SPILLWAY SECTION—continued

Capacity—Continued.
  Outlet gates (8 outlets), water elevation 1,034 ---- 38,200 cubic feet per second
Spillway bridge:
  Spans ---- 3 of 105 feet; plate girders encased in concrete, 30 feet center to center, 22-foot roadway, 4-foot 6-inch sidewalk.
  Structural steel ---- 772,199 pounds
  Concrete ---- 451 cubic yards
  Reinforcing steel ---- 78,067 pounds
Drum gates:
  Three gates, 100 by 14 feet, hinged at upstream edge. Bureau of Reclamation type. Top of gate shaped to crest of dam.
  Top of gate when raised ---- elevation 1,034
  Operation ---- By flotation in pit surrounding each gate; water supplied by gravity from reservoir; full lift possible with reservoir at elevation 1,022.
Weights:
  Structural steel ---- 905,000 pounds
  Hinge castings ---- 302,175 pounds
  Anchor bolts ---- 72,237 pounds
  Operating equipment, piping, etc ---- 185,876 pounds
Painting:
  Entire inside and outside of ends and bottom—1 coat Bitumastic regular primer and 1 coat Bitumastic regular enamel.
  Outside of top plate—1 coat Bitumastic 70–B primer and 1 coat Bitumastic 70–B enamel.
  Downstream surface—Bitumastic 70–B primer, brushed on cold.
Outlet gates:
  Size ---- Inside dimensions 5 feet 8 inches by 10 feet
  Number ---- 16—1 service and 1 emergency in each outlet
  Conduit liner ---- 48 linear feet semisteel each conduit, balance unlined concrete.
  Maximum operating head ---- 169 feet
  Maximum hydrostatic head ---- 182 feet
  Operation ---- Hydraulic cylinder, oil pressure. One 20-horsepower pump, capacity 15 to 20 gallons per minute. Maximum pump pressure 2,000 pounds per square inch; lifting speed 1 foot per minute.
  Velocity through gate ---- 80 feet per second
  Air vents ---- Two 30-inch round shafts; each one for 4 conduits
  Total shipping weight ---- 2,173,220 pounds
  Trashrack structures ---- 4 semicircular concrete towers, each with 50 openings 6 feet 2¼ inches by 10 feet. Total net area through racks 3,090 square feet for each tower. Provided with compressed air pipes with nozzles for cleaning.
Elevator:
  Capacity ---- 3,000 pounds live load
  Type ---- Automatic, push-button control
  Motors ---- 25-horsepower, 440 volts, 60-cycle, 3-phase
  Lift ---- 181.48 feet, speed 250 to 300 feet per minute
  Platform (outside dimensions) ---- 6 feet 9 inches by 5 feet 11 inches
  Purpose ---- To provide access to roadway and galleries at elevations 880, 996, and 1,051.

MISCELLANEOUS MECHANICAL EQUIPMENT

Deepwell sump pump:
  Type ---- 21 H Deepwell
  Capacity ---- 800 gallons per minute at 70-foot head
  Motor ---- 20-horsepower, type M. T., 3-phase, 440 volts, 60-cycle
Plunger pump:
  Type ---- 12-inch, plunger
  Capacity ---- 22 gallons per minute at 70-foot head
  Motor ---- 5-horsepower, 3-phase, 60-cycle, 440 volts

Dam—Continued

EARTH FILL DAM

Core wall:

Reinforced concrete 5 feet thick trenched to rock, wood sheathed, removed prior to concreting, 584 feet long, maximum height, 95 feet. Reinforcing, ⅞-inch round bars 9 inches center to center each face, each way. Excavation, 6,021 cubic yards; concrete, 5,988 cubic yards.

Embankment:

Length along centerline, 290 feet. Top elevation, 1,061.0. Top width, 36.5 feet. Greatest height, 91.0 feet. Upstream slope 3 to 1, riprapped. Downstream slope 3 to 1, sodded. Riprap, 30-inch rock, smooth surface on 12-inch gravel blanket—32,200 square feet. Clay placed loose in 6-inch layers, rolled by truck at 100 pounds per square inch. Volume, 71,608 cubic yards.

Test data (6 samples):

Moisture content	30 percent
Dry weight of material in place	83 pounds per cubic foot
Dry weight of laboratory compacted material at 30 percent moisture	92 pounds per cubic foot

Power Plant

INTAKES

One gate for each of 2 penstocks. Opening 16 feet 6 inches wide, 28 feet 6 inches high. Tractor type continuous roller trains with floating wedges for seating and unseating. Wedge movement, vertical 18 inches, horizontal ⅜ inch. Gate clearance or movement to seal $\frac{3}{16}$ inch.

Gate seats ______ Cast steel, stainless steel roller tracks (Brinnel hardness 277) bronze seals with 5¼-inch contact width. Load taken into concrete through tension and compression bolts, 108,000 pounds per linear foot of seat.

Rollers ______ Stainless steel 4 inches in diameter, 6½ inches long, 556 per gate. Theoretical load, 45,000 pounds per roller; elastic limit by test, 100,000 pounds per roller.

Hoist ______ Double drum, 64 inches in diameter, 14 parts, 1-inch cable from each drum to each set of sheaves on lifting beam. Cable continuous over equalizing sheave. Load per cable part, 8,574 pounds. Hoist motors, two 60 horsepower; hoisting speed, 7.6 feet per minute; lowering speed 8.0 feet per minute without power.

Controls____________ In hoist house; auxiliary lowering control in powerhouse

Weights (each gate):

Lifted parts	121.25 tons
Gate and hoist	162.2 tons
Seats and guides	28.8 tons

Trashrack structures ______ 2 semicircular concrete towers, each with 64 openings 5.8 by 10 feet. Total net area through racks, 3,030 square feet for each tower. Provided with compressed air pipes with nozzles for cleaning.

TRASHRACKS

Number installed	64 racks in each of 2 intake structures
Size of sections	12 feet 6 inches high by 7 feet 10 inches wide
Size of bars	6¼ by 1 inch
Spacing of bars	6 inches on centers
Total weight, racks, beams, and bolts	765,824 pounds

PENSTOCKS

Number____ 2

Type______Concrete transition section 20 feet long. Electric machine welded steel plate, 1⅛ to 1⅜ inches thick.

Dimensions______________________ 20 foot diameter; 153 feet 10 inches long

Weight installed____ 585.7 tons

Painting______Priming coat, Bitumastic primer brushed cold; final coat, Bitumastic regular enamel, brushed hot at 425° F.

Air vents____________________________ 1—24 inches diameter for each penstock

POWER PLANT—Continued

POWER STATION

Generating capacity (2 units) .... 112,000 kilovolt-amperes, 100,800 kilowatts, 132,000 horsepower.
Structural steel framework and reinforced concrete.
Principal dimensions:
Length .... 205 feet
Width .... 67.5 feet
Height (bedrock to parapet) .... 153 feet
Structural steel:
Use .... Complete steel frame
Weight .... 912.9 tons
Traveling crane:
Capacity .... Two 125-ton hooks with lifting beam; two 20-ton auxiliary hooks.
Characteristics:

	Motor	Speed	Minimum movement
	Horsepower	*Feet per minute*	*Inch*
Main hooks	60	4 to 5	1/16
Auxiliary hooks	60	25 to 35	....
Trolleys	10	25 to 30	1/4
Bridge	50	90 to 105	1/4

Shipping weights:
Machinery .... 267,000 pounds
Structural steel .... 178,000 pounds

TURBINES

Number 2
Capacity .... 2 at 66,000 horsepower each at 165-foot head
Spacing of turbines, center to center of units .... 60.0 feet
Vertical distance from center line of distributor to floor of draft tube .... 37.0 feet
Tested efficiencies:

	Unit 1	Unit 2
	Percent	*Percent*
Maximum combined efficiency turbine and generator	91.3	91.5
Maximum turbine efficiency, 180-foot head	93.2	93.3
Full-gate efficiency, 180-foot head	84.1	84.0

Type .... Francis right-hand vertical shaft, single runner, specific speed 48.8, oil lubricated babbit bearing, 36 by 34 inches.
Casing .... Steel plate, 1 11/16 to 1 inch, 430 tons, 40,000 rivets
Discharge per unit at full gate (field test) .... 4,560 cubic feet per second at 180-foot head, 4,350 cubic feet per second at 165-foot head.
Normal speed .... 112.5 revolutions per minute
Runaway speed .... 215 revolutions per minute at 207-foot head
Diameter of runner .... Inlet 161 inches; discharge 165.5 inches
Weights:
Runner .... 100,000 pounds each
Shaft .... 53,500 pounds each
Shaft .... Forged carbon steel, 35 inches diameter, 14 feet long, 6-inch diameter hole in center.
Hydraulic thrust .... 385,000 pounds
Total suspended load .... 550,000 pounds
Clearances:
Runner .... Total 0.08-inch top, 0.12-inch bottom
Wicket gates .... Total 0.025 inch
Bearing .... Total 0.014 inch

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POWER PLANT—Continued

TURBINES—continued

Leakage ________________ 0.9 percent maximum turbine discharge
Shipping weights:
Hydro machinery ________________ 1,550,000 pounds
Structural steel forms ________________ 950,000 pounds

DRAFT TUBES

Type ________ Concrete elbow, dividing into 3 discharge passages at lower end
Pit liner ________________ ⅝-inch steel, 9 feet 2 inches long
Throat diameter ________________ 13 feet 9½ inches
Horizontal length (center line of turbine to downstream face) ________ 57 feet
Net area at outlet opening ________________ 623 square feet

GENERATORS

Number ________________ 2
Rating ______ 56,000 kilovolt-amperes, 13,800 volts, 50,400 kilowatts, 0.9 power factor, 112.5 revolutions per minute, 3 phase, 60 cycles.
Type ________________ Alternating current, vertical shaft
Drive ______ 66,000 horsepower Francis turbine, 112.5 revolutions per minute
Bearings ____ Combination Kingsbury type thrust bearing and segmental guide bearing, below rotor, total load 1,296,000 pounds.
Rotor ______ Cast steel spider, 75,000 pounds; 64 poles, laminated ring assembled in field shrunk to spider at 66° C.; weight excluding exciters, 483,000 pounds; outside diameter, 27 feet 8 inches. Weight of rotor, 483,000 pounds; of shaft, 51,800 pounds; of exciters, 19,000 pounds.
Stator ______ Star connected, split winding with 6 main and 3 neutral terminals: total weight, 215,120 pounds: in four sections of 53,780 pounds each; 8 coils at each joint installed in field; outside diameter, 32 feet 6 inches.
Efficiencies:

	Percent load							
	0.9 power factor				1.0 power factor			
	100	75	50	25	100	75	50	25
Guaranteed efficiency, percent	97.300	97.000	96.100	93.400	97.750	97.450	96.600	94.100
Tested efficiency, percent [1]	97.968	97.698	97.012	94.729	98.249	97.997	97.368	95.293
Kilowatt rating	50,400	37,800	25,200	12,600	56,000	42,000	28,000	14,000

[1] Unit No. 2 tested.

Brakes ______ 16 units with asbestos faced shoes; air pressure, 80 pounds per square inch; 7½ minutes to stop from ½ speed.
Jacks ______ Hydraulic, using brakes with oil at 1,500-pound pressure from portable hand pump. Capacity, 678,000 pounds; lift 1½ inches.
Ventilation and cooling ________ Air circulation through 8 water cooled radiators within steel casing, 41 feet 8 inches diameter. Cooling water, 840 gallons per minute at 25° C. temperature. Air 100 cubic feet per minute. Surface capacity of coolers, 14,480 square feet.
Temperatures ______ Guaranteed maximum indicated temperature at rated kilovolt-ampere with ambient water at 25° C. Stator—100° C. as determined by detectors; field—100° C. as determined by resistance; slip rings—105° C. as determined by thermometer; cores and mechanical parts—90° C. as determined by thermometer.
Temperature detectors ______ 12 resistance type coils, each 10 ohms at 25° C. with 3 leads (6 used at present).
Maximum capacity ______ 64,400 kilovolt-amperes at a maximum safe indicated temperature of 120° C. as determined by detectors when ambient water is at 25° C.

POWER PLANT—Continued

GENERATORS—continued

Line-charging capacity------ 49,300 kilovolt-amperes maximum without becoming completely self-excited.
Synchronous condenser capacity---------------------- 40,000 kilovolt-amperes.
Synchronous condenser losses-------- 1,160 kilowatts total losses including exciters and rheostats when operated as condenser at rated volts and speed and synchronous condenser capacity.
Reactances------Direct axis synchronous reactance (X_d) 97.5 percent; quadrature axis synchronous reactance (X_q) 59 percent; direct axis transient reactance (X'_d) 34.4 percent; quadrature axis transient reactance (X'_q) 59 percent; direct axis subtransient reactance (X''_d) 23.2 percent; quadrature axis subtransient reactance (X''_q) 28 percent; negative sequence reactance (X_2) 26 percent; stator leakage reactance (X_1) 15.6 percent; zero sequence reactance (X_o) 23 percent.
Short circuit ratio--1.045
Symmetrical R. M. S. short circuit currents calculated by manufacturer: initial three phase 4.32×rated current; initial single phase 3.63×rated current; initial line to neutral 4.32×rated current; sustained three phase 2.19× rated current; sustained single phase 2.98×rated current; sustained line to neutral 4.32×rated current.
Full-load regulation------33 percent of 0.9 p. f. and 22 percent at unity p. f. of rated voltage at rated voltage and speed.
Field current------848 amperes at rated load and 0.9 p. f.; 653 amperes at rated load and 1.0 p. f.; 914 amperes as 40,000-kilovolt-ampere condenser, over-excited.
Damper windings------Low resistance, with connections between poles—extra cost $6,150 each generator.
Damper torque--------80 percent at 5 percent slip at normal terminal voltage
Flywheel effect (WR^2)----------------------------------70,000,000 pound-feet [2]
Main exciter:
- Type--------Shunt wound, interpole, separately excited from pilot exciter
- Rating--------------------------------275 kilowatts, 1,100 amperes, 250 volts
- Mounting----------------Vertical, mounted on generator shaft above rotor
- Response ratio--2.0
- Time response of voltage regulator--------------------------------3 cycles
- Ceiling voltage--550 volts

Pilot exciter:
- Type--Compound wound, self-excited
- Rating-----------------------------------15 kilowatts, 60 amperes, 250 volts

Exciter temperatures--------Maximum indicated temperature rise of main and pilot exciters above 40° C.; ambient air as determined by thermometer, based on rated amperes and volts; armature winding 35° C.; field winding 35° C.; commutator 50° C.; core and mechanical parts 35° C.
Neutral breaker:
- Type--- B-8-A De-ion grid
- Rating------Single pole, single throw, 600 amperes, 15,000 volts manually operated; 36,000 amperes interrupting capacity or 15,000 kilovolt-amperes at rated voltage.
- Mounting------Steel cubicle complete with 600 amperes, 15,000 volts, single-pole, single throw, disconnecting switches.

Neutral reactor:
- Type--"CL" air cooled
- Rating-------Reactance per specification 0.82 ohm, tests averaged 0.841 ohm. Current carrying capacity 8,000 amperes for one minute per specifications. Tests were on basis of 3,500 amperes for 5½ minutes which resulted in temperature rise of 71° C. by resistance obtained 30 seconds after shutdown. Ambient temperature of 24.5° C. Insulated for 15,000 volts.
- Mounting--------------------------Enclosed in safety type metal enclosure.

Cooling water temperature------Average temperatures of surface cooler intake water range from 50° F. with the reservoir level at 998 to about 55° F. at elevation 980.
Cooling water required------For thrust bearing oil, 60 gallons per minute at 25° C.

POWER PLANT—Continued

GENERATORS—continued

Fire extinguishing system:

Equipment.......Carbon dioxide equipment for two generators. The equipment consists of 2 banks of twenty 50-pound cylinders each, together with the appropriate piping, thermostats, valves, and control panel. Normal operation automatic but manually operated if desired.

Oil for bearings..........Recommendations by manufacturer: Viscosity of 250 seconds Saybolt at 145° F.

Oil capacity......................For thrust and guide bearings—750 gallons.

Weights......Generator, complete—1,200,000 pounds net. Heaviest piece (rotor)—500,000 pounds net.

Dimensions......Width, over-all (diameter) 41 feet 8 inches; height, above foundation 11 feet 2 inches.

Erection time....................Rotor, each 42 days; complete, each 90

GOVERNORS

Servomotors......................................450,000 foot-pound capacity

Operating pressure................................300 pounds per square inch

Minimum time to close gates...4 seconds

Motor driven fly-balls receiving power from separate permanent magnet generator on the main generator shaft.

Speed droop control...............................Adjustment 0 to 6 percent

MAIN POWER TRANSFORMERS

Number installed.................................... 6—in 2 banks (no spares)

Type......Single-phase, outdoor, self-cooled, oil-insulated, with inert gas seal, General Electric Co., type H.

Rating, each transformer......18,667 kilovolt-amperes continuous 55° C. rise; 110,000Y/154,000Y—13,200 volts; 60 cycles; subtractive-polarity.

Rating, each bank..................................... 56,000 kilovolt-amperes

Connections............Delta (L. V.) to Star (H. V.) with neutral grounded through reactor. Transformer may be operated either with neutral solidly grounded, or grounded through a reactor.

Tertiary connections.. None

Taps in H. V. winding......Two 2½ percent full capacity taps above and two below rated voltage (161700–157850–154000–150150–146300). Taps are changed only when transformer is out of service. Used for maintaining rated bus voltage.

Taps in L. V. winding... None

Losses......At 75° C. and 60 cycles, test average 6 transformers: No load and rated voltage—44.56 kilowatts; no load and 110 percent rated voltage—64.24 kilowatts; 75 percent load, unity p. f., and rated voltage (total)—81.42 kilowatts; 100 percent load unity p. f., and rated voltage (total)—108.79 kilowatts.

Exciting current...... 2.33 percent of normal full load current at rated voltage, test average 6 transformers.

Impedance........8.87 percent at 18,667 kilovolt-amperes, test average 6 transformers.

Regulation........ At temperature of 75° C. and rated kilovolt-ampere output and rated voltage at 60 cycles. Test average 6 transformers. 0.74 percent at power factor of 1.00; 4.49 percent at power factor of 0.90; 5.84 percent at power factor of 0.80.

Efficiencies...... Test average 6 transformers at 75° C. 1.0 power factor. 99.42 percent at full load; 99.42 percent at ¾-load; 99.35 percent at ½-load; 98.97 percent at ¼-load.

Insulation tests...... Low voltage winding—34 kilovolts; high voltage winding, line terminal to ground (induced potential)—323 kilovolts; high voltage neutral (grounded Y)—70 kilovolts.

Power Plant—Continued

MAIN POWER TRANSFORMERS—continued

Bushing tests:

	Kilovolts to ground 1-min.	Average wet flashover
	Kilovolts	*Kilovolts*
Low-voltage bushing	50	45
High-voltage bushing	410	355
High-voltage neutral bushing	90	85

Polarity gap settings------------ 13.2-kilovolt bushing—5½ inches; 154-kilovolt (grounded Y) bushing—4 feet 1⅛ inches; neutral bushing (rated 34.5 kilovolts)—11 inches.
Weights------Each transformer, approx. Core and coils—65,000 pounds; tank and fittings—33,000 pounds; oil (7,740 gallons)—58,000 pounds; total—156,000 pounds.
Dimensions ------ Floor space—13 feet 3¼ inches by 16 feet 9½ inches; height, over-all—24 feet 6 inches; height, over tank—18 feet 1¼ inches; headroom required for removing core and coils—32 feet 8⅝ inches.
Truck base------Supported by four 12-inch diameter wheels fitted with chromium-plated Timken roller bearings with Alemite fittings. Wheel base is 6 feet 1½ inches for track gage of 6 feet.

TRANSFORMER NEUTRAL REACTOR

Number installed--------------------------Two—1 for each transformer bank.
Type------Single-phase, outdoor, self-cooled, oil-insulated, General Electric Co. Neutral Impedor, Type CLSO.
Rating--------12,600 kilovolt-amperes, 21,000 volts, 60 cycles, designed to carry 600 amperes for 1 minute without exceeding 120° C. rise over 40° C. ambient for 3 phase, 154,000-volt circuit; reactance 35.0 ohms; resistance 0.440 ohm at 75° C.; impedance volts unit (1) 21,200, unit (2) 20,600; thyrite resistor across terminals.
Maximum 5 seconds current rating------1,200 amperes (R. M. S.) without mechanical or electrical injury.
Insulation------Insulation tests, winding to ground, 1 minute—34.0 kilovolts H. V. neutral bushing—34.5 kilovolts; grounding bushing—15.0 kilovolts.
Polarity gap settings------Neutral bushing—11 inches; grounding bushing—5½ inches.
Bushing current transformer------One 1200/5 amperes located in ground lead.
Weights------Each reactor approximate: core and coils—1,300 pounds, case and fittings—2,200 pounds, oil—2,200 pounds, total—5,700 pounds.
Dimensions---------Floor space—3 feet 10¾ inches × 5 feet 9 inches; height, over-all—7 feet 9¼ inches; height, over tank—6 feet ½ inch, headroom required for removing core and coils—10 feet 4 inches.
Base--------------------------------------For flat surface or pad mounting.

MAIN SWITCHBOARDS

Type------Benchboard for main control with 2 vertical back-to-back switchboards for instruments and relays. All panels of stretcher-leveled steel.
Number of panels------8 panels for benchboard, 14 panels for instrument board, 14 panels for relay board.
Size of panels------For benchboard, 20 inches wide; for vertical boards, 20 inches wide, 90 inches high.

STATION SERVICE AND UNIT AUXILIARY BOARDS

Type------Steel, totally enclosed type constructed of welded vertical steel frames, with each circuit breaker enclosed in an individual steel cell suitably lined with arc-resisting insulation.
Number of boards------Five, as follows: 1 common station auxiliary board; 1 unit auxiliary board (for 2 generators); 1 compressor cubicle; 1 oil purifier cubicle; 1 heater cubicle.

POWER PLANT—Continued

STATION SERVICE AND UNIT AUXILIARY BOARDS—continued

Current interrupters:

	Short-time rating of interrupters (R. M. S. amperes)		
	Type LX	Type W	Type K
1 second	40,000	20,000	10,000
¼ second	60,000	30,000	17,000
1/16 second	100,000	35,000	20,000
1/64 second	100,000	40,000	25,000

DIRECT-CURRENT SWITCHBOARD

Type______Same general type as station service and unit auxiliary boards, see above.

BATTERY

Type______9-plate cells
Number of cells______120
Ampere-hour capacity______320 amperes for 8-hour rate or a discharge rate of 40 amperes for 8 hours to a final voltage of 1.75 volts per cell.
Net weight per cell______195 pounds approximate

BATTERY M–G SETS

Type______Diverter pole generator set (motor-generator set)
Number______2
Rating______Generator—20-kilowatt, direct-current, 280-volt; motor—30-horsepower, 440-volt, 3-phase, 60-cycle, 1,750 revolution-per-minute induction.

STANDARD FREQUENCY SOURCE LOAD AND FREQUENCY CONTROL

Principal equipment______1 dynamic tuning fork; 1 motor-generator set, 1,000-watt (Type FB4–Y3, approximate size 48 inches long, 13¼ inches wide, 17¾ inches high); 1 master clock, 12-inch diameter for main switchboard; 1 time standard clock, 6-inch face; 1 secondary clock, 6-inch face; 1 time-error indicator, 6-inch face.

AUXILIARY POWER TRANSFORMERS

Number installed______2
Type______HT, self-cooled, outdoor type
Rating, each transformer______750-kilovolt-ampere, 3-phase, 60-cycle, 55° C. above ambient 40° C.
Connections______14,200 Y–460-volt Delta
Taps______13845/13490/13135/12780 volts, full capacity
Voltage tests______34 kilovolts on high voltage, 10 kilovolts on low voltage, both for 60 seconds.
Efficiencies______At 75° C. 1.0 p. f.: Full load—98.70 average from tests; ¾-load—98.82 average from tests; ½-load—98.83 average from tests.
Regulation______1.0 p. f.—1.088 average from tests; 0.9 p. f.—3.184 average from tests; 0.8 p. f.—3.89 average from tests.
Total losses______At 75° C.; full load—9,840 watts average from tests; ¾-load—6,750 watts average from tests.

MISCELLANEOUS ELECTRIC EQUIPMENT—CABLES

Main generator cable. 1/c, 1,500,000 cir mils, 91 strands, 21 kilovolts, insulated with 16/64 inch of best quality sulphate process wood pulp paper, approximately 40 percent of which is superdense and the remainder regular density paper. Insulated conductor is shielded with a 3-mil 13/16-inch-wide perforated aluminum tape, interlocked with manila tape; the shielded and insulated conductor is covered with 8/64-inch commercially pure lead sheath. Approximate outside diameter—2.26 inches; approximate net weight per foot—9.9 pounds; minimum bending radius—22.6 inches; guaranteed minimum

POWER PLANT—Continued

MISCELLANEOUS ELECTRIC EQUIPMENT—CABLES—continued

Main generator cable—Continued.
insulation resistance Megohm miles at 15.5° C.—66; guaranteed maximum safe temperature of copper—78° C.

Generator neutral cable. 1/c, 500,000 cir mils, stranded copper, 27/64 inches, varnished cambric insulation for 26 kilovolt; outside diameter, 1.773 inches; net weight, 2,841 pounds per 1,000 feet.

Auxiliary and control cable. Performite rubber insulation having at least 35 percent by weight of best grade new rubber. Covering of cable depends upon location and service. Braid covering with moisture-resisting flameproof braid is generally used in dry locations, lead covering in wet places, and a few special cases have armored cables and asbestos-covered cables.

MISCELLANEOUS MECHANICAL EQUIPMENT

Sump pump:
- Type — Type K Aurora Pump
- Capacity — 300 gallons per minute at 40-foot head
- Motor, 5-horsepower, 3-phase, 60-cycle, 440-volt, 1,750 revolutions per minute.

Air compressors:
- 330 cubic feet per minute compressor:
  - Capacity—free air — 330 cubic feet per minute
  - Piston displacement — 463 cubic feet per minute
  - Gage pressure — 100 pounds per square inch
  - Speed — 275 revolutions per minute
  - Cooling water — 17.5 gallons per minute
  - Motor — 75-horsepower, 1,115 revolutions per minute, 440-volt 3-phase, 60-cycle.
- 45 cubic feet per minute compressor:
  - Capacity—free air — 53 cubic feet per minute
  - Piston displacement — 65 cubic feet per minute
  - Gage pressure — 100 pounds per square inch
  - Speed — 350 revolutions per minute
  - Cooling water — 1 gallon per minute
  - Motor — 15 horsepower, 1,750 revolutions per minute, 440-volt, 3-phase, 60-cycle.
- 8 cubic feet per minute compressor:
  - Capacity—free air — 8.1 cubic feet per minute
  - Piston displacement — 11 cubic feet per minute
  - Gage pressure — 300 pounds per square inch
  - Speed — 490 revolutions per minute
  - Motor — 3-horsepower, 1,750 revolutions per minute, 440-volt, 3-phase, 60-cycle.

Oil purifiers and pumps:
- Type — Combination centrifuge and filter press
- Centrifuge—capacity — 1,200 gallons per hour at a temperature of 50° C.
- Centrifuge motor — 3-horsepower, 1,750 revolutions per minute, 440-volt, 3-phase, 60-cycle.

Dirty oil pump
- Suction lift — 15 feet
- Discharge pressure — 40 pounds per square inch
- Capacity — 1,400 gallons per hour
- Motor, 5-horsepower, 1,750 revolutions per minute, 440-volt, 3-phase, 60-cycle

Clean oil pump
- Suction lift — 15 feet
- Discharge pressure — 80 pounds per square inch
- Capacity — 1,500 gallons per hour
- Motor — Same motor as used on clean oil pump

Filter press — 1,200 gallons per hour

Oil heaters:
- Number of units — Two
- Capacity — 54 kw each

SWITCHYARD EQUIPMENT

OIL CIRCUIT BREAKERS

Type------FHKO-339-72B-F4 outdoor, triple-pole, single-throw, 250-volt direct current control, solenoid operated, trip-free, and fully automatic. Mounted on concrete pads.
Rating--------161-kilovolt 1,200-ampere 1,500,000 kilovolt-ampere interrupting capacity, O. C. O.+O. C. O. with operating interval of 15 seconds and recovery voltage of not less than 2,400 volts per microsecond.
Weights------One complete single-pole unit with oil—25,500 pounds; 1 complete single-pole unit without oil—13,700 pounds; oil per single-pole unit (1,550 gallons approximate)—11,625 pounds; weight of 1 bushing—1,350 pounds.

DISCONNECTING SWITCHES

Type------Outdoor, 3-pole, vertical-break, underhung; some manually operated and some motor operated; operation through rotating insulators.
Rating ---------------------------------- 161 kilovolts, 600 amperes continuous
Insulators -- Switch type
Tests, flashover------Single unit 163 kilovolts (dry), 111 kilovolts (wet); 4-unit stack 517 kilovolts (dry), 443 kilovolts (wet).
Tests, cantilever------Cap mounted failed at edge of cap at 1,230 pounds; base mounted failed at pin at 1,395 pounds.
Tests, torsion-----------------------Average failure at 62,320 inch-pounds.
Motors--------0.9/1.5 horsepower, 200/280 volts, 6.25-amperes full load, RC type and No. 2 frame.

BUSES AND CONNECTIONS

Bus size------------------- 3-inch standard I. P. S. hard-drawn copper tubing
Bus spacing---- 8 feet 6 inches between phases; 7 feet between phase and ground.
Bus insulators---------------------------------Porcelain; spaced 32 feet apart
Strain insulators:
Dry flashover test-------------------------------- 92 kilovolts each unit
Wet flashover test-------------------------------- 58 kilovolts each unit
Puncture test-------------------------------- 154 kilovolts each unit
Cable connections-------------------------------- 500,000 cir mils bare copper

LIGHTNING ARRESTERS

Type------Thyrite, outdoor, single pole, model No. 9LAID106, 12 units, grounded neutral. Set of 3 arresters for each bank of transformers.
Rating------Line to line—161 kilovolts normal, 169 kilovolts maximum; maximum permissible line to ground—145 kilovolts.
Clearances--------Minimum line to ground—4 feet 6 inches; minimum line to line—6 feet 4 inches.
Dimensions------Height over arrester only—15 feet 7¾ inches; vertical anchoring support directly above arrester consists of 4 Locke No. 7785 insulators and a spacer rod. Insulator stack is 4 feet 10 inches high and rod spacer is 2 feet 9 inches.
Weights------Net weight of arrester only—1,000 pounds; net weight of insulators—320 pounds.
Mounting---Flat surface type

TRANSFER CAR

Type------Structural steel 4-wheel type with 24-inch double-flanged wheels with Hyatt roller bearings. Wheels suitable for 100-pound rails.
Capacity--Designed for 100 tons
Wheel spacing-- 8-foot track gage
Track rails on top of car------------------------------ 6-foot track gage
Weight-------------------------------------- 16,500 pounds approximate

ELECTRIC ELEVATOR LIFT

Type------------------------------Structural steel 4-screw type, motor drive
Capacity --110 tons

Switchyard Equipment—Continued

electric elevator lift—continued

Lift.................27 inches in not more than 5 minutes
Motor.................15-horsepower, 440-volt, 3-phase, 60-cycle, 1,200 revolutions per minute.
Weight.................16,000 pounds

Construction Plant

cableways

Span.................1,925.5 feet
Sag.................77 feet±to 106 feet
Main cable.................3-inch locked coil type, area 6.53 square inches
Stress including impact.................65,500 pounds per square inch
Ultimate strength.................168,000 pounds per square inch
Counterweights:
- Head tower................. 390 tons
- Tail tower................. 445 tons

Operating data:
- Design load.................18 tons (plus 25 percent impact)
- Maximum load—spillway girder.................25 tons per cableway
- Lowering speed—full load.................400 feet per minute
- Hoisting speed—full load.................300 feet per minute
- Carriage travel—full load:
  - In.................1,200 feet per minute
  - Out.................1,600 feet per minute
- Traversing speed of towers.................50 feet per minute
- Concrete placing cycle.................2½ to 6 minutes

cement

Type B modified, 35 to 55 percent tri-calcium silicate; tri-calcium aluminate not more than 8 percent; average initial set, 3 hours 15 minutes; final set 5 hours 41 minutes; fineness 1,600 to 2,200 cm^2 per gram by Wagner turbidimeter.
Total cement used in concrete, including that used in dam, powerhouse, and roadway, grouting not included.................1,089,528 barrels
Grouting:
- Foundation proper.................203,054 cubic feet
- Reservoir rim.................257,736 cubic feet
- Buffalo Creek divide.................5,677 cubic feet
- (These include cement, rock flour, and small amounts of sand.)

concrete aggregates production

Material.................Knox dolomite, 2,071,000 tons; solid weight, 176 pounds per cubic foot; weight of screened aggregate in stock piles, about 90 pounds per cubic foot; removed by wagon drills and blasting in 30-foot lifts in 12- to 14-foot benches; maximum charge, 2,000 pounds dynamite.
Crushers:
- Primary................. 42-inch gyratory
- Secondary................. 5½-foot cone
- Sand plant................. 4 hammer mills, 42 by 48 inches

Capacity................. 325 tons per hour

concrete production

Mixing.................Three 3-cubic-yard Smith tilting mixers; drum speed, 11 to 12 revolutions per minute; mixing time, 2½ minutes in mixer; normal cycle, 3¼ minutes.
Plant capacity................. 180 cubic yards per hour
Placing.................To cableways via transfer cars; 6-cubic-yard buckets for mass concrete; 1 surface and 4 internal vibrators per crew. Depth of pour 5 feet. Minimum time between successive pours, 72 hours. Maximum concrete placed in 24 hours, 4,090 cubic yards; in 1 calendar month, 92,780 cubic yards.

Construction Plant—Continued

concrete production—continued

Tests------Six 17-inch cores taken from mass concrete averaged 5,110 pounds per square inch at 6 months; 6- by 12-inch laboratory cylinders averaged 4,404 pounds per square inch at 28 days, and 6,396 pounds per square inch at 6 months, and 6,876 pounds per square inch at 1 year. Temperature rise, mass, 35° F. Weight, mass, 159 pounds per cubic foot.

Loyston Dike

Material------Rolled clay fill

Purpose------To dam a low saddle near Loyston, Tenn.

Dimensions------Top elevation, 1,065; volume, 80,458 cubic yards; length, 1,980 feet; maximum height, 32 feet; top width, 10 feet; maximum base width, 170 feet; upstream slope, 3 to 1 riprapped; downstream slope, 2 to 1 sodded.

Placing------6-inch layers when compacted by sheepsfoot rollers at 1,000 pounds per linear foot, or trucks at 100 pounds per square inch.

Test data (7 samples):

- Moisture content------28 percent
- Dry weight of material in place------89 pounds per cubic foot
- Dry weight of laboratory compacted material at 28 percent moisture------96 pounds per cubic foot.

Drilling and grouting------Rock grouted to ground water level with single line of holes spaced 10 to 20 feet on centers.

Appendix B

REPORTS OF CONSULTING ENGINEERS AND GEOLOGISTS

The following reports contain the joint recommendations and approvals following studies by any two or more consultants. Individual reports of consultants working separately are not included because in most instances they are voluminous and cover detailed engineering or geologic work ordinarily done by members of the Authority's staff.

General Considerations, July 28, 1933

Considerations in building Cove Creek (Norris) Dam as agreed upon by engineering consultants, Mr. J. L. Savage and Mr. S. M. Woodward, and the geologic consultants, Mr. C. P. Berkey, Mr. Arthur Keith, and Mr. A. C. Swinnerton.

1. The dam is in approximately the right location.

2. Not enough hydrographic data has been collected to make a close determination of the best height within a range of, perhaps, 30 feet. Since it would take years to accumulate the necessary data, the best possible approximation will be made. Any variation in height will not affect the safety of the dam but only the balance of economy between extra storage on one hand and extra cost of land and of the cost of the larger dam on the other hand.

3. The dam probably will be of the gravity concrete type, but with the spillway over the dam itself, and not at the side. The cross section of the dam will probably be increased, with increased safety, but also with increased cost. There will be other modifications in design.

4. There probably will be some leakage of water under or around the dam, but not to a degree which will endanger the structure.

5. It would be impossible, without the expenditure of a very large amount of money, to be sure that there will be no leakage from the reservoir through the hills, but the prospect is not great enough to justify any hesitation in building the dam.

6. The Tennessee Valley Authority would not be justified in building the dam by the present somewhat rapidly prepared plans. It will require a force of about 40 designing engineers about 6 months to complete the plans. However, some of the preliminary work, such as building a highway and stripping the base of the dam, can begin sooner.

The plans will be prepared in the Denver office of the United States Bureau of Reclamation. This office has prepared plans for the Boulder Canyon Dam in Nevada, for the Madden Dam in the Panama Canal Zone, for over 20 big dams in the West and about 100 small ones. It is very much the largest dam-designing organization in America, if not in the world. The Boulder Canyon Dam is looked upon as one of the most accurately and carefully designed dams that ever has been built. Greater progress can also be made by using this organization than by creating a new organization of engineers.

7. The United States Army Engineers used good judgment in selecting the site of the dam and in deciding on the type of dam to be built.

Arthur E. Morgan, *Chairman*.

Sufficiency of Subsurface Explorations for the Foundation of the Concrete Dam and Character of the Foundation Rock, December 14, 1933

The Rock.—The foundation rock at this site is the Knox dolomite, a well-known formation occurring in heavy bedded strata and lying nearly flat or dipping gently downstream.

The whole formation has been thoroughly reorganized since its original deposition so that the rock is now compact and very hard. Furthermore, this transformation was accompanied by changes in composition and internal structure so that it is now a crystalline dolomite, and the shrinkage that resulted has caused the strata to develop an uneven or wavy and closely fitted bedding structure which adds materially to its strength in mass and greatly increases its resistance to deformation or sliding along the bedding surfaces.

The formation in general above ground water exhibits the usual irregularities of surface, including sinks and underground channels characteristic of such rock. These effects are clearly the result of weathering and percolating rain water over the very long time that the region has been exposed to erosion.

Below ground water, however, as is indicated by the borings and other exploration, the rock in general is sound and, as far as known, is virtually free from important solution effects.

Support.—Exploratory borings cover the whole length of the proposed concrete dam and there has been taken the additional precaution of sinking a 70-foot shaft with cross cut extending out beneath the bed of the river. These give adequate basis for judging this series of borings. These explorations and tests fully support the conclusion that the foundation rock beneath the main section of the dam is of excellent quality. Altogether we conclude that under normal construction treatment, including grouting, especially of any jointed areas, the foundation rock will be of ample strength and structural integrity to support the load which the dam will impose.

Leakage.—The river bed rock, which has been subjected to low percolation gradients, will, in our opinion, prove to be amply tight and free from leakage after grouting.

The abutment rock, where percolation gradients are high, has been considerably weathered and affected by solution channels and caves. It may be necessary to remove considerable rock at the abutments to reach sound material. That satisfactory material exists at reasonable depths is indicated by the rock cores and more especially by the ground water levels in the drill holes and the elevation of springs in the neighborhood. Isolated solution channels, such as the drain channels of old and now obliterated sink holes, at the abutments may be anticipated. Most of these will be sealed by grouting. If not, they may result in some leakage when the reservoir is first commissioned, but it should be possible to seal them afterwards. In any case, no danger to the structure is anticipated by such leakage.

Adequacy of exploration.—It is the opinion of the Board that the exploratory work already completed on the foundation and abutment rock is sufficient for present purposes.

CHARLES P. BERKEY.
F. W. SCHEIDENHELM.
S. M. WOODWARD.
L. F. HARZA.
J. L. SAVAGE.
CHAS. H. PAUL.

RESERVOIR CLEARING, DECEMBER 14, 1933

It is the opinion of the Board:

1. That the upper limit of the reservoir clearing should be established, for the time being, at elevation 1,020 (spillway crest).

2. That below elevation 960 (probable low water) only trash and loose branches need be cleared. Trees which grow below elevation 960, the tops of which extend above that height, should be felled or cut off so as not to project above water surface at that elevation.

CHARLES P. BERKEY.
L. F. HARZA.
F. W. SCHEIDENHELM.
J. L. SAVAGE.
S. M. WOODWARD.
CHAS. H. PAUL.

DEPARTURE FROM PREVIOUS DESIGNS, DECEMBER 14, 1933

The Board approves the following departures from previous designs:

Section of dam.—The cross-section of the dam has been thickened in the higher portions of the two abutment sections to provide the necessary stability against the lateral transfer of water load from the higher central portion of the dam by twist action.

Location and type of spillway.—The spillway has been located in the river channel section of the dam in order to deliver the water into its ultimate channel without lateral changes in the direction of flow. This arrangement permits the use of an hydraulic jump apron and results in the safest and most economical device for dissipating the energy of the spillway discharge without destructive erosion.

Spillway gates.—Drum gates have been adopted for the spillway crest in order to provide a long overflow type of gate with maximum head room for passing unavoidable flows of large drift during extraordinary floods.

Outlet gates.—The flood discharge conduits are provided with tandem rectangular slide gates operated from a gallery in the dam, except two conduits for close regulation of flow which are controlled by needle valves installed in the powerhouse. The rectangular slide gates have been adopted in order to deliver the flood control discharges into the river channel where the stream bed is protected by the concrete spillway apron. Neither needle valves nor free discharge butterfly valves can be used at this location on account of the flow over the spillway.

Trashrack for power plant.—The trashrack structure, in front of the power plant, has been extended to a height which will give access to the top of the rack structure for raking and replacement of rack sections at frequent intervals.

CHARLES P. BERKEY.
F. W. SCHEIDENHELM.
S. M. WOODWARD.
L. F. HARZA.
J. L. SAVAGE.
CHAS. H. PAUL.

GENERAL FEATURES OF DESIGN OF CROSS SECTION OF THE CONCRETE PORTION OF THE DAM, MARCH 17, 1934

It is the opinion of the Board that the concrete dam has been so designed as to provide ample stability under the various conditions of loading to which it will be subjected.

L. F. HARZA.
CHARLES P. BERKEY.
J. L. SAVAGE.
F. W. SCHEIDENHELM.
CHAS. H. PAUL.
S. M. WOODWARD.

SPECIAL ENGINEERING RESEARCHES, MARCH 17, 1934

The importance of this project is so great that careful consideration will be given to many questions on which existing engineering knowledge is not readily available. It will be necessary, therefore, for the engineering staff of the Authority to make numerous research studies and investigations.

The Board suggests that such necessary studies be carried out with care and thoroughness so that the results obtained, in addition to providing the information needed for the immediate local construction purposes, will be sufficiently comprehensive to be of value to the engineering profession as a whole in extending the limits of engineering knowledge.

It is hoped that the results of such researches may be made available to the profession as a whole.

L. F. HARZA.
CHARLES P. BERKEY.
J. L. SAVAGE.
F. W. SCHEIDENHELM.
CHAS. H. PAUL.
S. M. WOODWARD.

Omission of Barge Lift, March 17, 1934

It is the opinion of the Board that there is no necessity for the installation of a barge lift at the Norris Dam at this time.

L. F. Harza.
Charles P. Berkey.
J. L. Savage.
F. W. Scheidenhelm.
Chas. H. Paul.
S. M. Woodward.

Reservoir—Subsurface Water Studies, March 17, 1934

The Board is impressed with the great importance and success of the studies made by the local staff covering springs, seepages, underground water levels and circulation, and the location of sinks and caves. We commend the care and good judgment with which this work has been done. These studies and observations should be continued through the period of construction so as to form a reliable background of fact by which to judge results of putting the reservoir in operation and as a guide to such corrective treatment as may be required.

L. F. Harza.
Charles P. Berkey.
J. L. Savage.
F. W. Scheidenhelm.
Chas. H. Paul.
S. M. Woodward.

Flood Capacity and Operating Elevations, March 17, 1934

In the operation of the dam it is proposed to hold the top of power storage at elevation 1,020. Under these conditions the dam will afford substantially complete control to not more than bank-full capacity of the channel downstream for a run-off from the entire drainage area of 8 inches in 1 day, 8½ inches in 2 days, 9 inches in 3 days, or 10 inches in 5 days. Such run-off is well in excess of that of any storm within the range of probability except perhaps at extremely rare intervals. In our opinion this measure of flood control is feasible and reasonably adequate for the Clinch River.

As to safety of the concrete dam itself, we believe that, in view of the flood absorption capacity of the reservoir, the design provides adequate safety against any possible flood contingency.

L. F. Harza.
Charles P. Berkey.
J. L. Savage.
F. W. Scheidenhelm.
C. H. Paul.
S. M. Woodward.

Foundation Grouting and Drainage, March 17, 1934

The Board recommends the following general program for foundation grouting:

Blanket grouting at pressures from 50 to 100 pounds per square inch, as determined by deformation gages, or otherwise to be unquestionably safe against heaving of the foundation rock. The grout holes should be 3-inch holes, 20 to 30 feet deep, and spaced at intervals of 20 to 40 feet to suit rock conditions.

Cut-off grouting at pressures from 200 to 500 pounds per square inch, as determined by deformation gages, or otherwise to be unquestionably safe against heaving of the foundation rock. The grout holes should be 3-inch diameter diamond- or shot-drill holes, 60 to 100 feet deep, spaced from 5 to 10 feet apart to suit rock conditions.

A line of drain holes approximately 50 feet apart, adapted for observation purposes, located immediately downstream from the line of cut-off grout holes.

The drain holes should be 3-inch diamond-drill or shot-drill holes extending in general to depths 10 feet less than the cut-off grout holes, but with occasional holes extending to depths of 100 feet.

L. F. HARZA.
CHARLES P. BERKEY.
J. L. SAVAGE.
F. W. SCHEIDENHELM.
CHAS. H. PAUL.
S. M. WOODWARD.

FOUNDATION INSPECTION, JUNE 27, 1934

On June 22 and 23, Norris Dam was inspected by consultants C. P. Berkey, L. F. Harza, J. L. Savage, F. W. Scheidenhelm, and S. M. Woodward. Messrs. Carl A. Bock, W. R. Chambers, B. M. Jones, C. H. Locher, and A. E. Morgan were present on one or both of the days.

The inspection included primarily all exposed foundation, the 36-inch core holes and cores, the records of 5½-inch core drilling and wagon drilling, the drifts in the east abutment, and the excavation into the cavities in blocks 25, 26, and 27. The foundation rock and the two exposed cavities in the west abutment were also inspected.

It was the general opinion that, from present indications, a satisfactory treatment of the cavernous area in blocks 25, 26, and 27 can be accomplished by removing all unsound rock, following all fissures and solution channels until they are outside the foundation, and driving a drift near the upstream face for intercepting the seam on the floor of the next lower ledge, as well as the seam about 6 feet above this floor. If it becomes evident as this work progresses that this treatment would be almost as expensive as complete removal of the cavernous ledge, then it would probably be desirable to carry the entire excavation to the floor of the next lower ledge.

In the narrow locality of seamy and folded rock near the upstream face of the dam, in blocks 34 and 35, the foundation excavation has been carried about 8 feet deeper than over the surrounding area. It was recommended that this excavation be extended westward far enough to remove all unsound rock and to cut off the major seams. The extension could be in the form of an open trench, or a V-shaped notch, either of which would be line-drilled as protection to the rock left in place. The upstream line of the notch, approximately on the upstream face of the dam, would join the downstream line with an apex angle of about 45°.

The deeper foundation excavation was terminated at a vertical shoulder 110 feet south of the axis. This shoulder has been trimmed to a slope of about 30° from the vertical. It was proposed that the top edge of the shoulder be re-cut to a slope of about 45°, starting 4 to 6 feet back of the face. This shoulder was discussed as a possible origin of shrinkage cracks in the concrete, and the advisability of reinforcing the concrete at the shoulder to prevent such cracking was discussed. The Bureau of Reclamation will investigate the conditions and advise as to the necessity or quantity and location of steel reinforcement. It was mentioned that, as a matter of general practice, any sharp break or offset in the foundation should be carefully examined and considered as a possible origin for cracks, and that steel reinforcement should be put in if necessary to prevent the cracking.

It was pointed out by Colonel Scheidenhelm that it would be undesirable to leave foundation rock higher than elevation 811 anywhere along the upstream line within the spillway portion of the dam.

It was cautioned that special care should be taken in washing and grouting under the toe of the dam, the area of the principal stresses. A general approval was expressed covering the subsurface treatment of the foundation as it is being done and briefly described as follows: The area is explored by wagon drills, 5½-inch and 36-inch core drills; the seams are washed by reversals of a supply of water and air under about 30 pounds pressure; and finally the seams are grouted. The area is first marked into A and B patterns, each covering a 40-foot square with holes at 10-foot centers in both directions, as shown on the record drawings. The B patterns are not washed and grouted until the surrounding A patterns have been washed, grouted and

set. Across the valley bottom, between the upstream line of the dam and a line 110 feet downstream from the axis, the drilling, washing, and grouting are being carried out on the spacing of 10-foot centers and to a depth of 30 feet in order to intercept rather important seams. Elsewhere the operation is divided into two stages as an aid to the washing and to more closely cover the area. First, the area is drilled to a depth of 20 feet on A and B patterns; then after the area has been washed and grouted, first on the A and then on the B patterns, the procedure is repeated on C and D patterns using holes 40 feet deep placed midway between the 20-foot holes of the A and B patterns.

After discussing treatment of the tailrace downstream from the draft tubes, it was concluded that filling with concrete all depressions in the slope that would cause hydraulic disturbances would be satisfactory.

Colonel Scheidenhelm emphasized the necessity for a thorough cleaning of the top surfaces at horizontal joints in the dam, expressing the view that, particularly at the weather faces, the top surface of the concrete should be removed to a considerable depth to insure against disintegration at the joints exposed to the weather. Dr. Savage expressed a belief that the amount of concrete to be removed depends to a large extent on the quality of the concrete. He felt that under the conditions of good concrete at Boulder Dam a quarter-inch removal was sufficient. It is possible that concrete made with Norris Dam aggregates may require some different treatment in view of the large amount of rock dust that may be left in the sand.

BARTON M. JONES.

CABLEWAY FOUNDATION REPAIRS, MAY 9, 1935

An inspection of Norris Dam was made on May 8.

This visit occurred a few days after a cave-in which undermined a portion of the cableway track on the west abutment of the dam site.

The cave-in was apparently induced by the excavation of clay from solution channels in the rock at a level 180 feet below the cableway track. If this assumption is correct, then one or more of the "chimneys" from the lower caverns must extend up to the level of the cableway track.

Mr. Ross White described to the Board the proposed method of repairing the foundation of the cableway track, which involves the excavation of about 12,000 cubic yards of earth to uncover solid rock down to the chimney openings. and the placing of a heavy concrete slab or plug to close the opening and provide an adequate foundation. The present steel track structure can then be supported on concrete piers founded on rock or on the concrete slab. Core drilling northward under the remainder of the cableway track will be done to examine the rock foundation there.

The Board approves the general method of repairs as proposed by Mr. White.

We found the job as a whole to be in excellent shape and the progress highly satisfactory. The work is several months ahead of schedule. The concrete work appears to be excellent, both as to quality and as to methods of handling and placing.

L. F. HARZA.
C. H. LOCHER.
CHAS. H. PAUL.
J. L. SAVAGE.
S. M. WOODWARD.

ABUTMENT GROUTING, AUGUST 1, 1935

We have examined the available data and the proposed program for exploring and grouting the abutments of the Norris Dam.

We approve this program, which is a flexible one and may be modified to suit the situation as it is further developed.

CHARLES P. BERKEY.
CHAS. H. PAUL.
J. L. SAVAGE.
S. M. WOODWARD.

GROUTING, SEPTEMBER 21, 1935

In our field inspection today at Norris Dam we have reviewed the plans for cut-off and grouting at the east abutment and east abutment ridge, and have examined the excavation for the concrete cut-off under the earth section, and the clay seam intercepted at about elevation 965. We approve the plan to tunnel out and cut off this seam under the ridge, and we also approve the tentative plan to drill and grout this ridge, insofar as practicable, from the tunnel instead of from the surface as first proposed.

The proposed grout curtain at the west abutment, to continue upstream around the nose of the hill and up the gulch along about the 1.020 rock contour, is approved.

We strongly urge that these grouting programs be pushed vigorously, particularly at the lower elevations, so that they may be completed as far as possible before water in the reservoir is raised above its present elevation.

We suggest that study be given to the desirability of blanketing the upper portion of the gulch on the reservoir side with earth at the narrow portion of the east abutment ridge, and placing enough riprap or suitable slope protection on the blanket to hold it against sloughing and wave wash.

We inspected the earth fill and discussed the grouting at the Buffalo Creek divide, and are well satisfied with the work accomplished there and with the plans for completion.

CHARLES P. BERKEY.
J. L. SAVAGE.
S. M. WOODWARD.
CHAS. H. PAUL.

DRUM GATES, NOVEMBER 1, 1935

We have considered the method of erection of drum gates on the Norris Dam, and express the following opinion:

Comparative estimates as to cost which are supported by actual cost records for a similar installation indicate a substantial saving in favor of erection by the manufacturing company. This saving alone seems to us to warrant award of the contract, including erection, to the low bidder.

Even if the cost estimate favored erection by Government forces, we believe that it is worth several dollars a ton to the TVA to be able to concentrate all responsibility for manufacture, erection, contingencies, and final acceptance tests, on the one organization.

We, therefore, believe that under the circumstances the award of this contract, including erection, to the Virginia Bridge & Iron Co. is fully justified.

CHAS. H. PAUL.
J. L. SAVAGE.

GROUTING, DECEMBER 31, 1935

We have reviewed with engineers Dale and Lewis the progress of grouting at Norris Dam since our last visit on November 25.

We are impressed with the importance of completing certain critical sections of this work before the water in the reservoir is raised to heights which might interfere with the progress or the effectiveness of this grouting.

We recommend that the program be set up so as to give assurance that the following sections will be completed not later than March 31, 1936:

East abutment—Core wall grouting station $2 + 20$ to $8 + 90$, 10-foot spacing. Rim grouting station $0 + 0$ to east $15 + 0$, 50-foot spacing.

West abutment—Rim grouting station $0 + 0$ to west $9 + 0$, 20-foot spacing.

J. L. SAVAGE.
CHAS. H. PAUL.

Grouting, April 6, 1936

The members of the Consulting Board spent most of Monday, March 30, in inspection of the work at Norris Dam and reviewed with your engineers the details of the grouting work done to date, and the proposed completion of the original grouting program.

The general character of construction work on the dam and appurtenances appears to be excellent.

With water in the reservoir standing at about elevation 964, there are as yet no indications of seepage either under or around the dam. While this may be considered as presumptive evidence that the grouting has been effective at the lower elevations, sufficient time has not yet elapsed to afford a conclusive test.

The plan to continue the original grouting program to completion is approved, and it is recommended that an adequate reserve be held in the construction fund to be used as needed to care for any seepage that may develop as the water in the reservoir is raised to higher elevations.

L. C. Glenn.
L. F. Harza.
Chas. H. Paul.
Warren J. Mead.
J. L. Savage.

Construction, July 14, 1936

The Board spent the forenoon of July 13 inspecting the Norris Dam. This inspection included a general examination of the condition of the concrete, the general character of the gate and valve installations, the slight seepage into galleries in the dam, the progress in the installation of power plant machinery and progress in other features of the project.

It is the opinion of the Board that the construction of Norris Dam and power plant is satisfactory and that excellent progress has been made since the time of the last Board meeting.

Present progress in the installation of power machinery and equipment indicates that the plant may be ready to generate power sometime in August. We see no objection to starting the generating units as soon as the usual tests and adjustments are made.

Charles P. Berkey.
O. N. Floyd.
L. C. Glenn.
G. W. Hamilton.
Chas. H. Paul.
L. F. Harza.
C. H. Locher.
Warren J. Mead.
J. L. Savage.

Additional Office Space at Norris Power Plant, October 9, 1936

Consideration has been given to the need for additional office space and to the need for a first-aid room at Norris power plant.

An office room approximately 17 by 19 feet has been provided in Norris power plant. This affords adequate space for the three desks proposed by the Operating Department and it appears that there is no need for additional office space under present conditions. In the event the Norris power plant becomes a major distributing station requiring more personnel than now contemplated, additional office space would probably be required.

A first-aid room should be provided in the Norris power plant and it appears that the storeroom on the floor at elevation 866.5 could readily be fitted up and used for this purpose. Substitute storage space could be provided elsewhere.

It is the opinion of the Board that the existing office space at Norris power plant is adequate for all present needs, and it is recommended that the proposed construction of additional office space over the control room be deferred until the future operating program is more definitely defined and there is a definite need for it.

The question of suggested changes at Norris brings to our attention the importance of ascertaining what the original designers had in mind before changes are made.

After power plants and other property are turned over to the Operating Department any construction changes proposed by the latter should be considered by both the Construction and Design Departments. It is believed that the Operating Department should not undertake changes involving additional construction work but that such changes if authorized by the Chief Engineer should be made by the Construction Department.

It is our belief that the operating force should devote all its activities to operating and maintenance.

GEORGE W. HAMILTON.
WILLIAM F. UHL.
L. N. MCCLELLAN.

Appendix C

DESIGN STUDIES

Gravity analyses and twist studies for Norris Dam were prepared by the United States Bureau of Reclamation and the results verified by check analyses computed by the Tennessee Valley Authority. Structural data, list of analyses made, constants and assumptions used in the analyses, together with a brief outline of the results, have been given in chapter 3.

BUREAU OF RECLAMATION DESIGN STUDIES [1]

The results of the gravity analyses for the empty condition and for the different flood conditions are shown on figures 320 to 322, inclusive. The results of the analyses made by the engineers of the Authority are also shown on these exhibits by the figures in parentheses as a matter of comparison with the results obtained by the United States Bureau of Reclamation. Sliding factors and shear-friction factors of safety for the assumption of two-thirds uplift are shown at the downstream face of the cross section in the cases of full reservoir loading. Location of resultants with and without uplift, also for the concrete weight alone, are shown by the arrows at the different elevations on the cross section. Unit stresses at the faces of the dam, both vertical and parallel to the slopes of the faces, are tabulated at the right and left sides of the drawings. Unit water pressures at the upstream face of the dam are tabulated at the different horizontal sections analyzed, for the different assumed elevations of reservoir surface. Tail water pressures at the downstream edge of the base are shown below the column of vertical foundation pressures of the downstream face and designated by asterisks. Both water pressures and unit pressures are shown in pounds per square inch in all cases; elevations of reservoir surface at the upstream face of the dam and the corresponding elevations of tail water surface at the downstream face are shown on the cross section, but the surface curves of the spillway overflow are not shown. All stresses are positive unless otherwise noted.

EMPTY RESERVOIR

The results of the stress analyses for the empty condition of the reservoir are shown on figure 320. The maximum compressive stress during this condition of loading occurs at the upstream edge of the base, elevation 800, and amounts to 233 pounds per square inch vertical pressure or 243 pounds per square inch inclined stress in the case of the abutment section, and 216 pounds per square inch vertical pressure or 224 pounds per square inch inclined stress in the case of the spillway section. A small amount of tension is indicated at elevations 900 and 950 at the downstream face of the abutment section, the maximum value occurring at elevation 900 and amounting to 1.0 pound per square inch vertical stress and 1.6 pounds per square inch inclined stress. This tension, although negligible in amount, is purely theoretical inasmuch as the water probably never will be completely drained out of the reservoir after the project is once put into operation. No tension is indicated at the spillway section.

Notes for Figures 320 to 323, Inclusive

1. Unit weight of concrete=150 pounds per cubic foot.
2. All normal stresses are compressive except those preceded by a negative sign which are tensile.

[1] Houk, Ivan E., Design of Cross Section. Gravity Analyses and Twist Studies for Norris Dam, U. S. Bureau of Reclamation Technical Memorandum, No. 354, Nov. 11, 1933.

3. All shear stresses are positive and are caused by shear forces acting thus $\rightleftarrows$
4. Total water load carried by vertical cantilever.
5. * Normal tail water pressure at downstream face of dam.

6. S = Sliding factor, two-thirds uplift $= \dfrac{\text{Horizontal Force}}{\text{Weight} - \text{Uplift}}$

7. $Q = \dfrac{(W-U) \times C + T \times R_S}{H}$

Q = Shear-friction factor of safety, two-thirds uplift.
W = Weight.
U = Uplift.
C = Coefficient of internal friction.
T = Thickness.
R_S = Unit shear resistance.
H = Horizontal force.

8. Coefficient of internal friction = 0.65.
9. Unit shear resistance = 400 pounds per square inch.
10. Broken resultants (↘) are for reservoir water surface elevation 1,060. Solid resultants (↘) are for reservoir water surface elevation 1,034.
11. The third points are indicated at each section.

SPECIAL NOTES FOR FIGURE 323

1. Assumed acceleration of earthquake = 0.10 g.
2. Assumed period of vibration of earthquake = 1 second.
3. The effect of the increase in vertical water pressure due to earthquake is omitted in this analysis.
4. This analysis also assumes that tail water pressure and uplift pressure are unchanged by effect of earthquake.

HYPOTHETICAL FLOOD

Figure 320 shows that the maximum sliding factor at the abutment sections of the dam for this condition of reservoir loading, assuming the full water load carried by gravity action, occurs at the base of the maximum section, elevation 800, and amounts to 0.666. The shear-friction factor of safety at the same elevation amounts to 7.1, the minimum value for any elevation in the abutment section. The maximum compressive stress occurs at the downstream face of the dam at elevation 800 and amounts to 194 pounds per square inch vertical pressure or 289 pounds per square inch inclined stress parallel to face. The maximum shearing stress occurs at the same location and amounts to 122 pounds per square inch. No tension occurs at any location during this condition of loading. Vertical compressive stresses at the upstream face of the dam are greater than the unit water pressure at elevations above 900 and only slightly lower than the water pressures at elevations below 900.

Figure 320 shows that the sliding factors, shear-friction factors of safety, maximum compressive stress, and maximum shearing stress in the spillway section of the dam are very similar to those in the abutment section except that the sliding factor is somewhat higher just below the drum gate chamber, as would be expected.

MAXIMUM FLOOD

Figure 321 shows that the maximum sliding factor in the abutment section of the dam for this condition of reservoir loading, assuming the full water load carried by gravity action, occurs at the base of the maximum section, elevation 800, and amounts to 0.638. The shear-friction factor of safety at the same elevation amounts to 7.3, the minimum value for any elevation in the abutment section. The maximum compressive stress occurs at the downstream edge of the base, elevation 800, and amounts to 183 pounds per square inch vertical pressure or 273 pounds per square inch inclined stress. The maximum shearing stress occurs at the same location and amounts to 116 pounds per square inch. No tension occurred at any location during this condition of loading. Vertical compressive stresses at the upstream face of the dam are greater than the unit water pressures at elevations above 850 and only slightly lower than the water pressures at elevation 800.

Figure 321 shows that the sliding factors, shear-friction factors of safety, maximum compressive stress, and maximum shearing stress at the spillway section of the dam are very similar to those in the abutment section except that the sliding factor is somewhat higher just below the drum gate chamber, as in the case of the hypothetical flood condition.

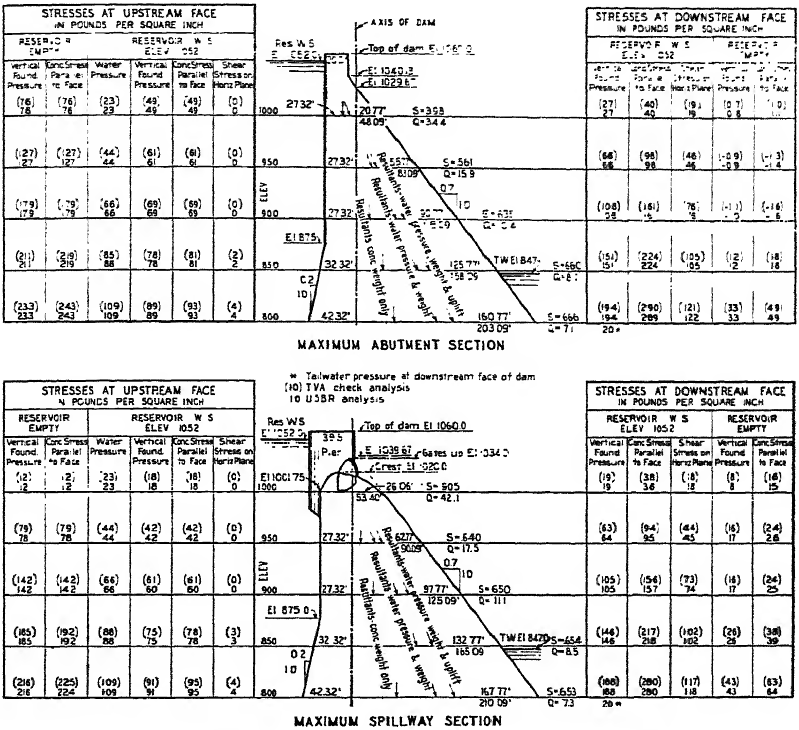


FIGURE 320.—*Gravity analysis—Reservoir elevation 1,052.*

NORMAL FULL LOAD OPERATION

The results of the stress analyses of the conditions of normal full reservoir operation are shown on figure 322.

Figure 322 shows that the maximum sliding factors in the abutment section for this condition of reservoir loading, assuming the full water load carried by gravity action, occur at the base of the maximum section elevation 800 and amount to 0.565. The shear-friction factor of safety at the same elevation amounts to 8.1, minimum value at any elevation in the abutment section. The maximum compressive stress occurs at the downstream edge of the base, elevation 800, and amounts to 159 pounds per square inch vertical pressure or 236 pounds per square inch inclined stress. The maximum shearing stress at the same location amounts to 102 pounds per square inch. No tension occurs at any location during this condition of loading. Vertical compressive stresses at the upstream face of the dam are greater than the unit water pressures at all elevations.

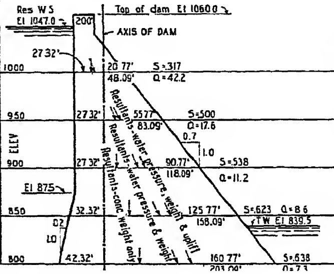
Figure 322 shows that the results of the analyses for the spillway section are very similar to those obtained in the abutment section.

HYPOTHETICAL SUPERFLOOD

The results of the stress analyses for the conditions which would exist in case of a hypothetical superflood, reservoir water surface at top of dam, elevation 1,060, are shown on figure 322.

STRESSES AT UPSTREAM FACE IN POUNDS PER SQUARE INCH

Reservoir empty: Vertical Found. Pressure	Reservoir empty: ConcStress Parallel to Face	Reservoir W S Elev 1047: Water Pressure	Reservoir W S Elev 1047: Vertical Found. Pressure	Reservoir W S Elev 1047: ConcStress Parallel to Face	Reservoir W S Elev 1047: Shear Stress on Horiz Plane	Elev
76	76	20	56	56	0	1000
127	127	42	70	70	0	950
179	179	64	80	80	0	900
211	219	86	88	91	-0.4	850
233	243	107	99	103	+1.7	800



STRESSES AT DOWNSTREAM FACE IN POUNDS PER SQUARE INCH

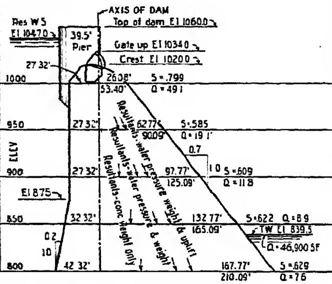
Elev	Reservoir W S Elev 1047: Vertical Found. Pressure	Reservoir W S Elev 1047: ConcStress Parallel to Face	Reservoir W S Elev 1047: Shear Stress on Horiz Plane	Reservoir empty: Vertical Found. Pressure	Reservoir empty: ConcStress Parallel to Face
1000	20	30	+14	0.8	1.1
950	56	84	+40	-0.9	-1.4
900	98	146	+68	-1.0	-1.6
850	140	209	+98	12	18
800	183	273	+116	33	49
	17*				

MAXIMUM ABUTMENT SECTION

* Tailwater pressure at downstream face of dam

STRESSES AT UPSTREAM FACE IN POUNDS PER SQUARE INCH

Reservoir empty: Vertical Found. Pressure	Reservoir empty: ConcStress Parallel to Face	Reservoir W S Elev 1047: Water Pressure	Reservoir W S Elev 1047: Vertical Found. Pressure	Reservoir W S Elev 1047: ConcStress Parallel to Face	Reservoir W S Elev 1047: Shear Stress on Horiz Plane	Elev
12	12	20	18	18	0	1000
78	78	42	45	45	0	950
142	142	64	66	66	0	900
185	192	86	82	85	0.7	850
216	224	107	98	102	1.8	800



STRESSES AT DOWNSTREAM FACE IN POUNDS PER SQUARE INCH

Elev	Reservoir W S Elev 1047: Vertical Found. Pressure	Reservoir W S Elev 1047: ConcStress Parallel to Face	Reservoir W S Elev 1047: Shear Stress on Horiz Plane	Reservoir empty: Vertical Found. Pressure	Reservoir empty: ConcStress Parallel to Face
1000	17	32	16	8	15
950	59	88	41	17	26
900	99	147	69	17	25
850	138	206	97	26	39
800	179	267	114	43	64
	17*				

MAXIMUM SPILLWAY SECTION

FIGURE 321.—*Gravity analysis—Reservoir elevation 1,047.*

Figure 322 shows that the maximum sliding factors in the abutment section for this condition of reservoir loading, assuming the full water load carried by gravity action, occurs at elevation 850 and amounts to 0.732. The minimum value of shear-friction factor of safety occurs at the base of the maximum section elevation 800 and amounts to 6.8. The maximum compressive stress occurs at the downstream edge of the base, elevation 800, and amounts to 212 pounds per square inch vertical pressure and 316 pounds per square inch inclined stress. The maximum shearing stress occurs at the same location and amounts to 130 pounds per square inch. No tension occurs at any location during this assumed extreme condition of loading. Vertical compressive stresses at the upstream face of the dam are slightly lower than the unit water pressures at all elevations below 1,000, the minimum ratio of the unit water pressure occurring at the base of the dam, elevation 800, and amounting to 0.646.

Figure 322 shows that the results of the analyses of the spillway section for this condition of reservoir loading are very similar to those obtained in the abutment section.

EARTHQUAKE EFFECTS

The effects of the assumed maximum earthquake shock on the stresses and stability of the structure during the empty and normal full operating condition of the reservoir are shown on figure 323. Sliding factors during the empty

condition of the reservoir will be 0.10 at all elevations in both spillway and abutment sections, since the horizontal acceleration during the occurence of the earthquake has been assumed to be one-tenth of gravity.

Figure 323 shows that the maximum compressive stress in the abutment section of the dam during the empty condition of the reservoir occurs at the upstream edge of the base, elevation 800, and amounts to 279 pounds per square inch inclined pressure, an increase of 36 pounds per square inch due to the earthquake effects. Some tension occurs at the downstream face, the maximum occurring at elevation 900 amounts to 40 pounds per square inch.

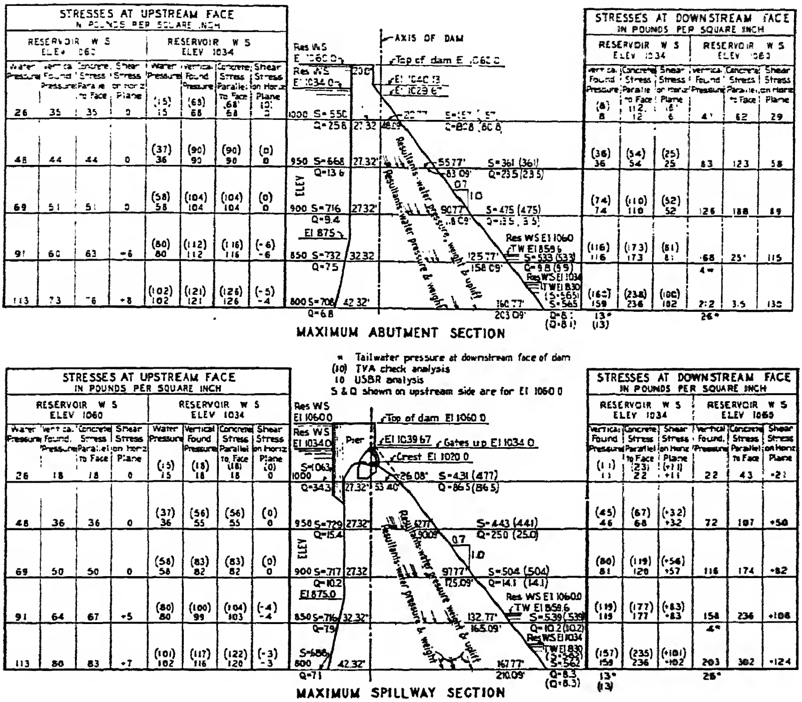


FIGURE 322.—*Gravity analysis—Reservoir elevations 1,060 and 1,034.*

Shearing stresses caused by the occurrence of the earthquake shock during the empty condition of the reservoir are relatively small, the maximum amounting to 54 pounds per square inch and occurring at the upstream edge of the base. The maximum shearing stress at the downstream face during this condition of loading amounts to 19 pounds per square inch and occurs at elevation 900.

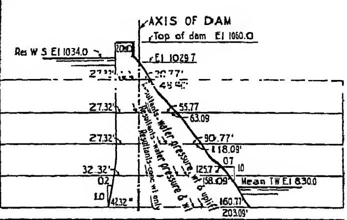
The analysis for the occurrence of the earthquake during the normal full operating condition of the reservoir shows that the maximum compression occurs at the downstream edge of the base, elevation 800, and amounts to 315 pounds per square inch inclined stress, an increase of 79 pounds per square inch due to earthquake effects.

No tension occurs at any location during this condition of loading. Vertical stresses at the upstream face of the dam exceed the increased unit water pressure at elevations above 900. At the face of the dam the inclined stress at the upstream face amounted to 71 pounds per square inch or 64.5 percent of the unit water pressure as increased by the earthquake shock. The maximum shearing stress during this condition of loading occurs at the downstream

edge of the base and amounts to 139 pounds per square inch. The maximum value of the sliding factor and the minimum value of the shear-friction factor of safety occurred at the same location, the former amounting to 0.759 and the latter to 6.0.

RESERVOIR EMPTY

Elevation	Vertical force (pounds) Weight	Resultant moment (foot-pounds) I	Resultant moment (foot-pounds) II	Conc. stress par. to face of dam Upstream I	Upstream II	Downstream I	Downstream II	Shear stress on horiz. plane Upstream I	Upstream II	Downstream I	Downstream II
1000	(264,540) 264,540	(-20,779,300) [illegible]	(-27,22[illegible]) [illegible]	(76) 76	(87) [illegible]	(1.0) [illegible]	(-15) -5	(0) [illegible]	(0) 0	(+0.5) +0.5	(-8) -8
950	(756,470) 756,470	(-10,628,000) 10,629,000	(-13,725,000) 13,725,000	(127) 127	(146) 146	(-1.3) -1.4	(-29) -29	(0) 0	(0) 0	(-0.6) -0.6	(-14) -14
900	(1,511,000) 1,510,900	(-30,098,000) 30,086,000	(-38,753,000) 38,741,000	(179) 179	(205) 205	(-1.6) -1.6	(-40) -40	(0) 0	(0) 0	(-0.8) -0.7	(-19) -19
850	(2,537,300) 2,537,200	(-59,447,800) 59,433,000	(-78,099,000) 78,064,000	(219) 219	(251) 251	(18) 18	(-28) -28	(-42) -42	(-48) -48	(+9) +9	(-13) -13
800	(3,891,800) 3,891,600	(-99,123,000) 99,261,000	(-133,740,000) 133,840,000	(243) 243	(279) 279	(49) 49	(-3) -3	(-47) -47	(-54) -54	(+23) +23	(-1) -1



RESERVOIR FULL - WATER SURFACE ELEVATION 1034

Elevation	Water pressure lbs. per sq. in. I	II	Horizontal force (pounds) I	II	Vertical force (pounds) Weight	Weight & Uplift	Resultants moment (foot-pounds) I	II	Sliding factor I	II	Shear-friction factor of safety I	II	Conc. stress par. to face of dam Upstream I	Upstream II	Downstream I	Downstream II	Shear stress on horiz. plane Upstream I	Upstream II	Downstream I	Downstream II
1000	(15) 15	(18) 18	(36,125) 36,125	(73,100) 73,100	(264,540) 264,540	(230,480) 230,480	(+1,869,270) +1,869,100	(+872,540) +872,890	(.157) .157	(.317) .317	(80.8) 80.8	(39.9) 39.9	(68) 68	(54) 54	(12) 12	(34) 33	(0) 0	(0) 0	(+6) +6	(+16) +16
950	(37) 36	(42) 42	(220,500) 220,500	(337,000) 337,000	(756,470) 756,470	(611,090) 611,060	(+4,454,300) +4,455,000	(-13,300) -14,400	(.361) .361	(.551) .552	(23.5) 23.5	(15.4) 15.4	(90) 90	(63) 63	(54) 54	(94) 94	(0) 0	(0) 0	(+25) +25	(+44) +44
900	(58) 58	(85) 85	(561,125) 561,125	(794,530) 794,530	(1,510,900) 1,518,900	(1,181,300) 1,181,200	(+5,034,500) +5,022,400	(-8,033,600) -8,046,600	(.475) .475	(.673) .673	(13.5) 13.5	(9.5) 9.5	(104) 104	(65) 65	(110) 110	(168) 168	(0) 0	(0) 0	(+52) +52	(+79) +79
850	(80) 80	(87) 87	(1,058,000) 1,058,000	(1,444,200) 1,444,200	(2,590,300) 2,590,300	(1,984,900) 1,984,800	(-1,340,50[illegible]) -1,352,1[illegible]	(-29,742,[illegible]) -29,754,[illegible]	(.53[illegible]) .533	(.728) .728	(9.9) 9.8	([illegible]2) [illegible]2	([illegible]6) 6	[illegible]	(173) 173	(244) 243	[illegible]	[illegible]	(-81) -81	[illegible]
800	(102) 102	(110) 110	(1,883,000) 1,883,000	(2,265,200) 2,262,100	(4,095,700) 4,095,500	(2,978,900) 2,978,500	(-18,977,000) -18,375,000	([illegible]1,215,[illegible]) [illegible]	(.56[illegible]) .565	(.76[illegible]) .759	(8.1) 8.1	(6.0) 6.0	(36) 36	[illegible]	(238) 238	([illegible]6) [illegible]5	[illegible]	(-8) -8	(+00) +132	(+139) +139

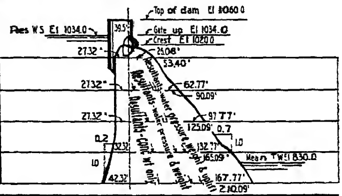
13* 13*
(13) (13)

GRAVITY ANALYSIS ABUTMENT SECTION

* Tailwater pressure at downstream face of dam
I Without earthquake effect
II With " "
(10) TVA check analysis
10 USBR analysis

RESERVOIR EMPTY

Elevation	Vertical force (pounds) Weight	Resultant moment (foot-pounds) I	Resultant moment (foot-pounds) II	Conc. stress par. to face of dam Upstream I	Upstream II	Downstream I	Downstream II	Shear stress on horiz. plane Upstream I	Upstream II	Downstream I	Downstream II
1000	(75,050) 75,604	(+147,050) +151,270	(-203,000) -198,960	(12) 12	(13) 13	(16) 15	(14) 14	(0) 0	(0) 0	(+8) +7	(-7) -7
950	(619,360) 619,170	(+4,106,900) +5,112,300	(+7,782,000) +7,566,900	(71) 71	(88) 87	(24) 26	(12) 13	(0) 0	(0) 0	(+12) +12	(+5) +6
900	(1,426,300) 1,426,100	(-23,111,000) -23,425,000	(+30,400,000) +30,364,000	(142) 142	(160) 159	(24) 25	(-2) -1	(0) 0	(0) 0	(+11) +12	(-1) -1
850	(2,525,100) 2,504,800	(-52,131,000) -51,140,000	(+68,514,000) +68,[illegible]81,000	(192) 192	(219) 218	(38) 39	(1) 2	(-37) -37	(-42) -42	(+18) +18	(0) +1
800	(3,9[illegible]2,000) 39[illegible]1,700	(-9,156,000) -9,511,000	(+124,[illegible]43,000) +123,810,000	(225) 224	(256) 256	(63) 64	(18) 18	(-43) -43	(-49) -49	(+30) +30	(+9) +9



RESERVOIR FULL - WATER SURFACE ELEVATION 1034

Elevation	Water pressure lbs. per sq. in. I	II	Horizontal force (pounds) I	II	Vertical force (pounds) Weight	Weight & Uplift	Resultants moment (foot-pounds) I	II	Sliding factor I	II	Shear-friction factor of safety I	II	Conc. stress par. to face of dam Upstream I	Upstream II	Downstream I	Downstream II	Shear stress on horiz. plane Upstream I	Upstream II	Downstream I	Downstream II
1000	(15) 5	(18) 18	(36,125) 36,125	(54,251) 55,135	(1[illegible]3,450) 1[illegible]470	(75,620) 73,840	(+230,500) +225,830	(+33,960) +23,190	(.477) .491	(.717) .749	(86.5) 86.5	(57.6) 56.7	(18) 18	(15) 15	(23) 22	(29) 28	(0) 0	(0) 0	(+11) +11	(+14) +14
950	(37) 36	(42) 42	(220,500) 220,500	(323,300) 324,200	(655,700) 655,040	(499,100) 497,380	(-1,118,000) +880,280	(-1,835,400) -2,227,600	(.441) .443	(.648) .652	(25.0) 25.0	(17.0) 17.0	(56) 55	(41) 39	(67) 68	(91) 92	(0) 0	(0) 0	(+32) +32	(+43) +43
900	(58) 58	(85) 85	(561,125) 561,125	(786,060) 786,980	(1,463,700) 1,462,000	(1,114,400) 1,112,600	(+430,000) +131,280	(-10,083,000) -11,067,000	(.504) .504	(.706) .707	(14.1) 14.1	(10.1) 10.1	(83) 82	(53) 52	(119) 120	(164) 165	(0) 0	(0) 0	(+56) +57	(+77) +77
850	(80) 80	(87) 87	(1,058,000) 1,058,000	(1,442,000) 1,442,900	(2,565,300) 2,548,300	(1,963,200) 1,961,400	(-5,910,000) -6,421,800	(-32,144,000) -32,715,000	(.539) .539	(.735) .735	(10.2) 10.2	(7.5) 7.5	(104) 103	(62) 61	(177) 177	(236) 237	(-4) -4	(+6) +6	(+83) +83	(+111) +111
800	(102) 102	(118) 118	(1,883,000) 1,883,000	(2,267,200) 2,265,100	(4,153,100) 4,151,580	(2,998,000) 2,996,000	(-21,761,000) -22,182,000	(-71,173,000) -72,156,000	(.562) .562	(.757) .756	(8.3) 8.3	(6.2) 6.2	(122) 120	(72) 71	(235) 236	(306) 307	(-3) -3	(+8) +8	(+101) +102	(+135) +135

13* 13*
(13) (13)

GRAVITY ANALYSIS SPILLWAY SECTION

FIGURE 323.—*Gravity analysis—Including earthquake effects.*

Maximum stresses, sliding factors, and shear-friction factors of safety in the spillway section of the dam are similar to those in the abutment section except that the tension at the downstream face during the empty condition of the reservoir is much lower in the spillway section than in the abutment section,

and the sliding factor during the full normal operating condition of the reservoir is somewhat higher in the spillway section than the abutment section in the case of the high elevations.

TRIAL LOAD TWIST ANALYSIS

Figures 324 and 325 show the results of the trial load twist analysis for hypothetical flood conditions; that is, reservoir water surface elevation at 1,052. Sliding factors at elevations above the base of the cantilever elements were calculated in the usual way; that is, by dividing the total horizontal water load carried by gravity action by the total weight of the concrete plus the

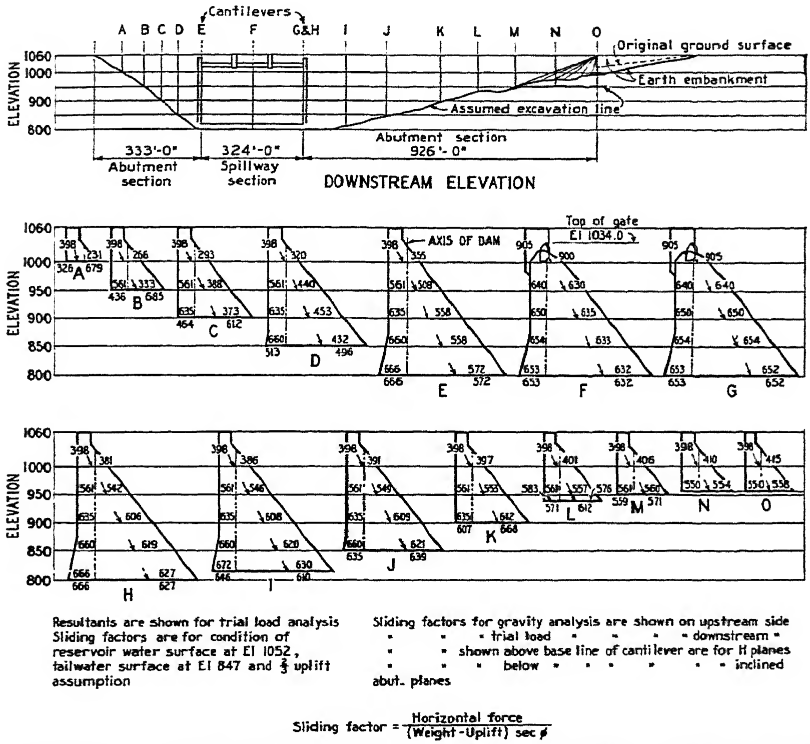


FIGURE 324.—*Trial load twist analysis—Location of cantilevers and beams—Sliding factors and resultants.*

weight of the water on the horizontal projection of the upstream face minus the total uplift force. Sliding factors at the base of the cantilever elements are calculated by dividing the total horizontal force, including the load brought in horizontally by twist action, by the component of the total vertical force acting in a direction normal to the slope of the base, properly corrected for uplift. This method makes proper allowances for the horizontal water load transmitted to the abutment by twist action and for the total area of the base of the cantilever along which sliding would take place if such movements should occur.

Fifteen cantilever elements and five horizontal elements were carefully analyzed. The cantilever elements were distributed along the profile so as to give

data at locations where the load distributions and deflections were most needed, as shown by previous trial load analyses of twist effects. The horizontal elements were located at 50-foot elevations from elevation 850 to elevation 1,000, and at elevation 1,060, the top of the dam in the abutment sections.

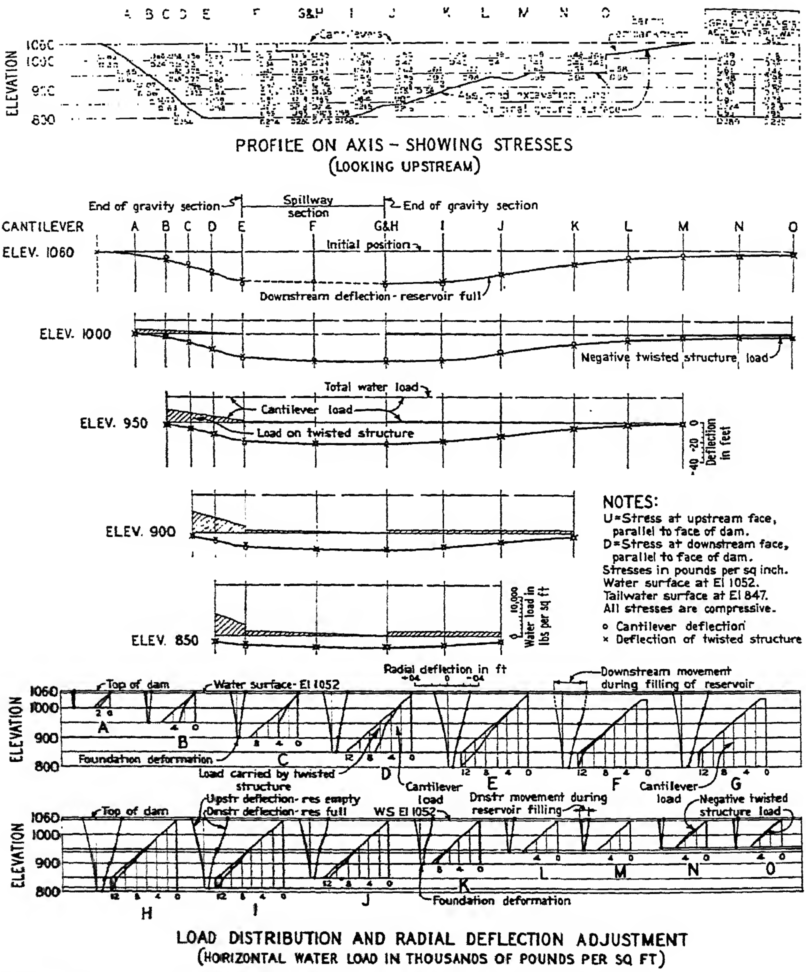


Figure 325.—*Trial load twist analysis—Load distribution and adjustments on horizontal elements—Reservoir full stresses and load distribution.*

Load distribution curves on figure 325 show that very little load was carried by twist action in the upper part of the dam, that a small amount was carried by twist at the lower elevations in the spillway and left abutment sections, and that appreciable portions of the water load were carried by twist action near the bases of the cantilevers in the right abutment section.

EFFECTS OF TWISTS ON SLIDING FACTORS

Figure 324 shows that the general effect of twist action on sliding factors was beneficial especially in the higher sections of the dam. For instance, twist effects at cantilevers E and H reduced the sliding factors at the foundation levels from 0.666 to 0.572 and from 0.666 to 0.627, respectively. Twist effects at cantilever I reduced the sliding factors at the foundation levels from 0.672 to 0.630 at the horizontal plane and from 0.646 to 0.610 at the inclined base. In the spillway section, which has the same base elevation as cantilevers E and H, the sliding factors were unchanged or somewhat reduced by making proper allowances for twist effects.

In the left abutment section the sliding factors were decreased appreciably at the higher cantilevers and increased slightly at the lower cantilevers near the end of the dam. However, increases were not serious inasmuch as the limiting value of 0.65 was not reached at any location except the base of cantilever K where the factor was increased from 0.607 to 0.668.

In the right abutment section the sliding factors were decreased at cantilever D and at the higher elevations at cantilevers B and C and were increased somewhat at the foundation levels of cantilevers A, B, and C, the limiting value of 0.65 being exceeded slightly at the bases of cantilevers A and B.

The reduction in sliding factors at cantilevers E and H and at the higher elevations in the other cantilevers of the abutment sections caused by twist effects was one of the principal reasons why it was possible to reduce the thickness of the dam at the abutment sections.

EFFECT OF TWIST ON STRESS

The stress profile in the upper part of figure 325 shows that the general effect of twist action on the distribution and magnitude of cantilever stress was beneficial but was not of great importance in either spillway or abutment sections. As a general rule stresses in the abutment sections were increased at the upstream face of the dam and decreased at the downstream face. Stresses in the spillway section were generally affected in the same way, but the changes were much smaller than in the abutment sections. Compressive stresses were not changed to tensile stresses at any location in either section. Since the maximum compressive stresses occur at the downstream edge of the base in both spillway and abutment sections, the effect of twist action was to reduce the magnitude of the maximum stresses.

EFFECT OF UPLIFT ON SLIDING FACTORS

Table 102 shows the effect of variations in uplift on the value of the sliding factor at different elevations in the abutment and spillway sections of the dam for the normal full operating condition of the reservoir.

TABLE 102.—*Effect of variations in uplift on sliding factor*

SLIDING FACTOR

Elevation	No U. A.		U. A. 1/4		U. A. 1/3		U. A. 1/2		U. A. 2/3		Full U. A.	
	A	S	A	S	A	S	A	S	A	S	A	S
1,000	0.137	0.324	0.143	0.371	0.146	0.390	0.151	0.435	0.157	0.491	0.169	0.660
950	.291	.337	.314	.370	.322	.383	.341	.411	.361	.443	.410	.527
900	.371	.384	.404	.422	.417	.436	.444	.468	.475	.504	.552	.598
850	.408	.408	.448	.449	.462	.464	.495	.499	.533	.539	.629	.643
800	.411	.405	.458	.453	.476	.471	.517	.512	.565	.562	.695	.696

U. A.—Uplift assumption. A—Abutment section. S—Spillway section.

Values of the sliding factor are tabulated for conditions of no uplift, one-fourth uplift, one-third uplift, one-half uplift, two-thirds uplift, and full uplift. These designations mean that the assumed uplift pressure curve, varying from full reservoir pressure at the upstream face of the dam to zero or tail water pressure at the downstream face, was applied to one-fourth, one-third, one-half, two-thirds, and the full horizontal area at each elevation, respectively.

Table 102 shows that for this condition of reservoir operation the sliding factor would not exceed 0.70 at any elevation at either section of the dam, even though the uplift pressure curve, through some unforeseen contingency, might be applied to the full horizontal areas. The occurrence of such an unforeseen contingency is not considered possible.

EFFECTS OF UPLIFT ON SHEAR-FRICTION FACTORS OF SAFETY

The effect of variations in uplift on the value of the shear-friction factor of safety at different elevations in the abutment and spillway sections of the dam for the normal full operating condition of the reservoir is shown in table 103.

TABLE 103.—*Effect of variations in uplift on shear-friction factor*

SHEAR-FRICTION FACTOR OF SAFETY

Elevation	No U. A.		U. A. ¼		U. A. ⅓		U. A. ½		U. A. ⅔		Full U. A.	
	A	S	A	S	A	S	A	S	A	S	A	S
1,000	81.4	87.2	81.2	86.9	81.1	86.8	81.0	86.6	80.8	86.5	80.5	86.1
950	23.9	25.5	23.8	25.3	23.7	25.2	23.6	25.1	23.5	25.0	23.3	24.8
900	13.9	14.5	13.7	14.4	13.7	14.3	13.6	14.2	13.5	14.1	13.3	13.9
850	10.2	10.6	10.1	10.4	10.0	10.4	9.9	10.3	9.8	10.2	9.6	10.0
800	8.5	8.8	8.4	8.6	8.3	8.6	8.2	8.5	8.1	8.3	7.9	8.1

U. A.—Uplift assumption. A—Abutment section. S—Spillway section.

Values of the shear-friction factors of safety are tabulated for conditions of no uplift, one-fourth uplift, one-third uplift, one-half uplift, two-thirds uplift, and full uplift.

Table 103 shows that the factor of safety against failure by sliding, for this condition of reservoir loading, would be 7.9 at the base of the abutment section and 8.1 at the base of the spillway section, even though the uplift pressure curve, through some unforseen contingency might be applied to the full horizontal area, a contingency which is not considered possible as stated above.

CHECK ANALYSES

GRAVITY ANALYSIS

The Tennessee Valley Authority made check computations on the gravity analyses originally computed by the United States Bureau of Reclamation. In these analyses two conditions of loading (namely, reservoir water surface at elevation 1,052.0 and tail water at elevation 847.0, and reservoir water surface at elevation 1,034.0 and tail water at elevation 830.0) were used. Additional analyses were made to determine the effect of earthquake action on the structure for the condition of reservoir water surface at elevation 1,034.0 (top of gates) and tail water at elevation 830.0. The results of these computations closely checked those made by the Bureau of Reclamation. The results of both analyses are shown on figures 320, 321, and 322.

The structural data used in the check calculation of the stability analysis are the same as were used by the Bureau of Reylamation in the original gravity analysis. The sections investigated were the abutment and spillway sections under loadings as assumed by the Bureau of Reclamation, the shapes of sections being the same as shown on figure 320.

These calculations show the method and equations[1] used by the Tennessee Valley Authority in checking the results of the Bureau of Reclamation stability analysis.

Abutment section.—Figure 320 shows the general outline of the abutment section and the planes along which stress calculations were made, together with tables showing the results of check analysis (shown in parentheses) and values obtained by the Bureau of Reclamation.

[1] Larsen, R. T., Stress and Stability Analyses of Straight Gravity Dams, U. S. Bureau of Reclamation Technical Memorandum No. 441, Feb. 28, 19

The concrete weights above various elevations, together with the eccentricity of the resultant as measured from the center of the base, are shown in table 104.

Vertical stresses at the upstream face and the downstream face were calculated from the equations:

$$S_{vu}=\frac{V}{144b}\left[1+\frac{6e}{b}\right]=\text{Vertical stress at the upstream face in pounds per square inch} \quad (1)$$

and

$$S_{vd}=\frac{V}{144b}\left[1-\frac{6e}{b}\right]=\text{Vertical stress at the downstream face in pounds per square inch} \quad (2)$$

In these equations V is the vertical component of the forces acting on the base being investigated, b is the length of the base, and e is the eccentricity of the vertical component V, or the distance from the center of the base to the point where the resultant of the forces passes through the base. Stresses parallel to the faces are computed by the equation:

$$S_p=S_v \operatorname{Sec}^2\phi \quad (3)$$

Where S_{pu} and S_{pd} are the stresses parallel to the faces $S_{pu}=S_{vu} \operatorname{Sec}^2\phi_u$ or $S_{pd}=S_{vd} \operatorname{Sec}^2\phi_d$ and ϕ is the angle the face makes when the verticle.

ϕ upstream = angle whose tangent = 0.2 = 11° 19′
Sec ϕ_u = 1.0198 Sec$^2\phi_u$ = 1.0400
(at elevations 800 and 850)

ϕ downstream = angle whose tangent = 0.7 = 35° 00′
Sec ϕ_d = 1.2208 Sec$^2\phi_d$ = 1.4904

In considering the case of empty reservoir, V in the above equations is equal to the weight of concrete above the section being investigated. Table 105 shows the resulting stresses for this condition. Plus (+) stresses are compression and minus (−) stresses are tension.

For the condition of reservoir full (elevation 1,052.0) the vertical stresses are the summation of the stress due to the water pressure, plus the stress due to weight of concrete as shown in table 105. In this case it is necessary to take into consideration both the horizontal and vertical water pressures acting on the face of the dam as well as the horizontal and vertical water pressures due to the tail water (elevation 847.0).

The vertical stresses at the upstream face and the downstream face due to the horizontal water pressure are computed by the summation of moments about the center of the base of the section. This summation of moments about the center of the base solved for the stress "S_v" results in the equation:

$$S_v=\pm\frac{wh^3}{144b^2} \quad (4)$$

Where S_v is the stress in pounds per square inch (minus at the upstream face and plus at the downstream face) w is the unit weight of water (62.5 pounds per cubic foot), h is the height of the water column above the base in feet, and b is the length of the base in feet at the elevation being investigated.

At elevation 850.0 there is additional stress due to the vertical pressure of the water on the incline section of the upstream face between elevation 850.0 and elevation 875.0, which results in a uniform compression stress over the entire base, plus stresses due to the amount of the vertical water forces. Stresses due to this vertical water load are expressed by the equation:

$$S_v=\frac{w'}{144b}\left[1\pm\frac{6e}{b}\right]=\frac{w'}{144b}\pm\frac{6w'e}{144b^2} \quad (5)$$

Where w' is the weight of the water acting vertically, e is the lever arm (eccentricity) of w' from the center of the base, and b is the base. This stress is plus at the upstream face and minus at the downstream face.

The stress due to water pressure at elevation 800.0 is the algebraic summation of stresses due to reservoir water pressure and tail water pressure. Stresses due to reservoir water pressures are calculated from the summation of equations

(4) and (5), (minus at the upstream face and plus at the downstream face). Stresses due to the tail water (elevation 847.0) are calculated from the same equation (plus at the downstream face and minus at the upstream face). Stresses parallel to the face are calculated from equation (3).

Table 106 shows the stresses at upstream and downstream faces in the case of reservoir full (elevation 1,052.0).

The eccentricity of the resultant water and concrete loads is expressed by the equation:

$$\frac{M}{W} \tag{6}$$

Where M is the summation of moments about the center of the base and W is the summation of concrete and water weights above the section. Resultant eccentricities are shown in table 107.

Horizontal shear stresses at the upstream and downstream faces are calculated by the equations:

$$S'_u = -(S_{vu} - P_u) \text{ Tan } \phi_u \tag{7}$$

$$S'_d = +(S_{vd} - P_d) \text{ Tan } \phi_d \tag{8}$$

Where

S'_u = Horizontal shear stress at upstream face.
S'_d = Horizontal shear stress at downstream face.
S_{vu} = Vertical stress upstream.
S_{vd} = Vertical stress downstream.
P_u = Unit pressure at the upstream face.
P_d = Unit pressure at the downstream face.
ϕ_u = Angle of upstream face with vertical.
ϕ_d = Angle of downstream face with vertical.

These shear stresses are given in table 108.

Uplift pressures are assumed to act in a straight line from full reservoir pressure at the upstream face of the dam to zero pressure at the downstream face, or to tail water pressure at locations where the plane being analyzed is below the elevation of the tail water surface. On this basis the uplift pressures are assumed to act over two-thirds the horizontal area of the concrete sections analyzed. At elevations above the base the uplift pressures are assumed to act in the pores of the concrete as well as along the plane of contact between the concrete and the foundation rock. The uplift pressure is expressed by the equation:

$$U = -\left[\frac{2}{3}\right]\left[\frac{whb}{2} + \frac{wh'b}{2}\right] \tag{9}$$

In this equation h' is the tail water head.

The sliding factor at the various elevations is therefore equal to

$$SF = \frac{\text{Horizontal Force}}{\text{Weight} + \text{Uplift}} \tag{10}$$

Table 109 shows the sliding factors based on the uplift assumptions stated above.

The eccentricity of the resultant of all loads (water, concrete, and uplift) is calculated from equation (6) in which the summation of all moments and the summation of all loads are used. Results of these calculations are shown in table 110.

The shear-friction factor of safety, with assumed two-thirds uplift, is equal to

$$Q = \frac{(\text{Weight} - \text{Uplift}) \times 0.65 + (144 \times 400b)}{\text{Horizontal Force}} \tag{11}$$

In this equation the constant 0.65 is the coefficient of internal friction and the constant for unit shear resistance is 400 pounds per square inch. These factors of safety are shown in table 111. Values for weight, uplift, and horizontal force are taken from table 109.

Spillway section.—The calculation of stresses for the spillway section is similar to the procedure followed in calculating stresses for the abutment section,

with the exception of the calculation of stresses due to the horizontal water pressure acting on the upstream face of the dam.

Figure 320 shows the general outline of the spillway section and the planes through which stress calculations were made, together with tables showing the results of the check analysis (shown in parentheses) and values obtained by the Bureau of Reclamation.

The weights of concrete (plus weight of gates) and the eccentricity of the forces are shown in table 112. In this calculation the opening for the gates is deducted. Vertical stress calculations for the condition reservoir empty are made from equations (1) and (2) and stresses parallel to the faces are calculated from equation (3). The summary of the stresses is shown in table 114.

$\text{Sec}^2\,\phi_u = 1.0400$ (elevations 800 and 850).
$\text{Sec}^2\,\phi_d = 2.0392$ (elevation 1,000).
$\text{Sec}^2\,\phi_d = 1.4904$ (elevations 950, 900, 850, and 800).

For the condition of reservoir full (elevation 1,052.0), the stress due to horizontal water pressure are calculated by taking moments about the center of the base of the section being investigated which reduces to the equation:

$$S_v = \pm\left[\frac{6w(h-h_1)^2}{144b^2}\right]\left[\frac{h_1}{3}+\frac{h}{6}\right] \qquad (12)$$

where h_1 is the height of the water column above the gates or 18 feet. This equation results in minus (—) stresses at the upstream face and plus (+) stresses at the downstream face.

Additional stresses are introduced by the weight of the water acting vertically on the top of the dam. Computations of this water load and moments are shown in table 113.

In the determination of the eccentricity of the resultant water and dead loads the moment due to water pressure on the upstream face is expressed by the equation:

$$M=\left[\frac{w(h-h_1)^2}{2}\right]\left[\frac{h}{3}+\frac{2h_1}{3}\right] \qquad (13)$$

In computing the sliding factors based on the uplift assumptions previously stated, the horizontal force acting on the upstream face is determined by the equation:

$$F=\frac{wh^2-w(h_1)^2}{2} \qquad (14)$$

The weight acting on the base being investigated is the summation of the concrete load plus the vertical water load. Tables 114 to 120, inclusive, give the computations and results of the stress analysis for the spillway section.

EARTHQUAKE EFFECTS

For the purpose of analyzing Norris Dam for stability against earthquake, it was assumed that the maximum earthquake shock would occur at time of full load operating conditions (elevation 1,034.0). The period of vibration was assumed as one second and the acceleration as one-tenth of gravity, the direction of vibration being at right angles to the axis of the dam. The full water load was assumed to be carried by gravity action. For the reservoir water surface at elevation 1,034.0, the corresponding tail water is at elevation 830.0.

The same method[2] of analysis was used by the Bureau of Reclamation in the original calculations and by the Tennessee Valley Authority in making a check analysis.

Abutment section.—The section used in this analysis is the same as used in the gravity analysis and is shown in figure 320. The weights of concrete are shown in table 104. Figure 323 shows the comparison of the values obtained by the Tennessee Valley Authority (shown in parentheses) and the Bureau of Reclamation.

[2] Westergard, H. M., Water Pressures on a Dam During Earthquakes, Transactions of American Society of Civil Engineers, vol. 98, 1933, pp. 418–433.

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For the condition of reservoir with water surface at elevation 1,034, and without earthquake effect, the equations for calculating stresses are the same as previously used in making computations with the condition of water surface at elevation 1,052. Table 121 shows the stresses due to water pressure and concrete combined with no earthquake effects. Tables 122 to 125 show the values of eccentricity of resultant water and dead loads, horizontal shear stresses, sliding factor, and shear-friction factor of safety, respectively, for this condition.

For the condition of reservoir empty the effect of maximum earthquake shock is assumed when the direction of foundation acceleration is downstream, i. e., when the inertia force of the dam caused by the earthquake is acting in an upstream direction.

From the assumptions previously stated

$$\frac{\text{Maximum Horizontal Acceleration of Foundations}}{\text{Acceleration Due to Gravity}}=0.1 \qquad (15)$$

then the horizontal inertia force "G" of the concrete or dead load weight above a section will equal

$$G=0.1\,V \qquad (16)$$

and correspondingly, the moment due to the earthquake force will be the moment of G about the center of the base. In this case the moments will be positive, i. e., G is acting in an upstream direction.

Computations of the resulting moment M_E (moment due to earthquake effect), together with the resulting eccentricity e of the resultant force, are shown in table 126.

The vertical stress at the faces is expressed by the equations:

$$S_{vuE}=\frac{V}{144b}+(M+M_E)\left[\frac{C}{I}\right]=\frac{V}{144b}\left[1+\frac{6e_E}{b}\right] \qquad (17)$$

$$S_{vdE}=\frac{V}{144b}-(M+M_E)\left[\frac{C}{I}\right. \qquad (18)$$

Stresses parallel to the face are expressed by the equation:

$$S_{puE}=S_{vuE}\,\mathrm{Sec}^2\phi_u \text{ and}$$
$$S_{pdE}=S_{vdE}\,\mathrm{Sec}^2\phi_d \qquad (19)$$

Table 127 shows the resulting computation and stresses.

Shear stresses on a horizontal plane are shown by the equations:

$$S'_{uE}=S_{vuE}\,\mathrm{Tan}\,\phi_u=0.2S_{vuE} \qquad (20)$$

$$S'_{dE}=S_{vdE}\,\mathrm{Tan}\,\phi_d=0.7S_{vdE} \qquad (21)$$

Values for these equations are shown in table 128.

In considering the effect of earthquake with high water elevation 1034.0, it is assumed that the maximum effect of the shock occurs when the direction of foundation acceleration is upstream, i. e., when the inertia force of the dam and water caused by the earthquake is acting in a downstream direction. In these calculations, the effect of vertical acceleration is ignored.

Earthquake in effect results in a parabolic increase in water pressure at the upstream face. This change in water pressure is expressed by the equation:

$$P_E=CK\sqrt{zh} \qquad (22)$$

Where C is a coefficient depending on the ratio of z to N, z equals distance from water surface to base of cantilever element and N equals period of horizontal vibrations of foundation—assumed to be one second. The value of C is expressed by the equation:

$$C=\frac{51.0 \text{ lb. per cu. ft.}}{\sqrt{1-0.72\left(\frac{z\times\text{sec.}}{1{,}000\,N\times\text{ft.}}\right)^2}} \qquad (23)$$

In equation (22), K is the ratio of maximum horizontal acceleration of the foundation to the acceleration due to gravity. For these computations the ratio is assumed as 0.1.

The horizontal inertia force of the water above any section is expressed by the equation:

$$P_{wE}=(2/3)CKh\sqrt{zh} \tag{24}$$

and the moment of P_{wE} about the center of gravity by the equation:

$$M_{wE}=(4/15)CKh^2\sqrt{zh} \tag{25}$$

The vertical stresses at the upstream and downstream faces are expressed by the equations:

$$S_{vuE}=\frac{\text{Summation } V}{144b}+(\text{Summation } M+M_E+M_{wE})\frac{6}{144b^2} \tag{26}$$

and

$$S_{vdE}=\frac{\text{Summation } V}{144b}-(\text{Summation } M+M_E+M_{wE})\frac{6}{144b^2} \tag{27}$$

The horizontal force in table 133 includes horizontal inertia force of water due to earthquake effect as expressed by the equation (22) and the horizontal inertia force of concrete expressed by the equation (16).

Spillway section.—The section used in this analysis is the same as used in the gravity analysis and shown in figure 320. The weights of concrete are shown in table 112.

Figure 323 shows the comparison of values obtained by the Tennessee Valley Authority (shown in parentheses) and the United States Bureau of Reclamation.

The moments due to the inertia of the concrete together with the moment of the concrete and the resulting eccentricity are shown in table 142.

The stresses at the faces are computed from equations (15), (16), and (3) and the results shown in table 143.

Computations of stresses, eccentricities, horizontal shear stresses, sliding factors, and shear-friction factors of safety are shown in tables 145 to 149, inclusive.

TABLE 104.—*Abutment section—computation of concrete weights, moments, and eccentricity—without earthquake effect*

Elevation			Area A square feet	Arm X feet	Moment A×X	Concrete weight A×150 pounds	Eccentricity e
1,060–1,040.13	20×19.87	=	397.4	25.00	9,935	----------	----------
1,040.13–1,000	20×40.13	=	802.6	25.00	20,065	----------	----------
	40.13×(28.09/2)	=	563.6	44.36	25,001	----------	----------
Total above elevation 1,000		=	1,763.6	[1] 31.19	55,001	264,540	7.86
1,000–950	48.09×50	=	2,404.5	39.05	93,896	----------	----------
	(35/2)×50	=	875.0	74.76	65,415	----------	----------
Total above elevation 950		=	5,043.1	[1] 42.50	214,312	756,465	14.05
950–900	83.09×50	=	4,154.5	56.55	234,937	----------	----------
	(35/2)×50	=	875.0	109.76	96,040	----------	----------
Total above elevation 900		=	10,073	[1] 54.13	545,289	1,510,950	19.92
900–850	118.09×50	=	5,904.5	74.05	437,228	----------	----------
	(35/2)×50	=	875.0	144.76	126,665	----------	----------
	(5/2)×25	=	62.5	13.33	833	----------	----------
Total above elevation 850		=	16,915	[1] 65.62	1,110,015	2,537,250	23.43
850–800	158.09×50	=	7,905.0	89.05	703,940	----------	----------
	(35/2)×50	=	875.0	179.76	157,290	----------	----------
	(10/2)×50	=	250.0	6.67	1,668	----------	----------
Total above elevation 800		=	25,945	[1] 76.08	1,972,913	3,891,750	25.47

[1] Arm X of the resultant weight above any elevation equals the summation of the moments (A×X) divided by the total area above the elevation since the section is of unit thickness.

For these calculations the reference line for moments is taken 15 feet upstream from the axis of the dam. Concrete weights are for a section 1 foot thick.

Eccentricity e is the distance from the center of the base to the point where the resultant acts.

TABLE 105.—*Abutment section—stresses—reservoir empty—without earthquake effect*

Elevation	V Lb.	b Ft.	e Ft.	$\frac{V}{144b}$	$\frac{6e}{b}$	S_{un} Lb./sq. in.	Stress parallel to upstream face S_{pu}	S_{ud} Lb./sq. in.	Stress parallel to downstream face S_{pd}
1,000	264, 540	48. 09	7. 86	38. 2	0. 981	+75. 7	+75. 7	+0. 7	+1. 0
950	756, 465	83. 09	14. 05	63. 2	1. 015	+127. 3	+127. 3	−. 9	−1. 3
900	1, 510, 950	118. 09	19. 92	88. 9	1. 012	+178. 7	+178. 7	−1. 1	−1. 6
850	2, 537, 250	158. 09	23. 43	111. 5	. 889	+210. 6	+219. 0	+12. 4	+18. 4
800	3, 891, 750	203. 09	25. 47	133. 1	. 752	+233. 2	+242. 5	+33. 0	+49. 2

TABLE 106.—*Abutment section—stresses—reservoir elevation 1,052.0—without earthquake effect*

UPSTREAM FACE

Elevation	Vertical stresses (lb./sq. in.) due to—						Conc. weight	Total vertical stress	Stress parallel to face
	Reservoir water pressure			Tail water pressure					
	Hor.	Vert.	Mom.	Hor.	Vert.	Mom.			
1,000	−26. 4	0	0	0	0	0	+75. 7	+49. 3	+49. 3
950	−66. 7	0	0	0	0	0	+127. 3	+60. 6	+60. 6
900	−109. 3	0	0	0	0	0	+178. 7	+69. 4	+69. 4
850	−143. 1	+2. 6	+7. 5	0	0	0	+210. 6	+77. 6	+80. 7
800	−168. 4	+6. 9	+19. 1	+1. 1	+1. 7	−4. 4	+233. 2	+89. 2	+92. 7
DOWNSTREAM FACE									
1,000	+26. 4	0	0	0	0	0	+0. 7	+27. 1	+40. 4
950	+66. 7	0	0	0	0	0	−. 9	+65. 8	+98. 1
900	+109. 3	0	0	0	0	0	−1. 1	+108. 2	+161. 3
850	+143. 1	+2. 6	−7. 5	0	0	0	+12. 4	+150. 6	+224. 1
800	+168. 4	+6. 9	−19. 1	−1. 1	+1. 7	+4. 4	+33. 0	+194. 2	+289. 5

TABLE 107.—*Abutment section—eccentricity of resultant water and dead loads—reservoir elevation 1,052.0—without earthquake effect*

Elevation	Summation M (ft.-lb.)	Summation W (lb.)	Eccentricity (ft.)
1,000	+615, 600	264, 540	+2. 33
950	−425, 920	756, 465	−. 56
900	−6, 483. 220	1, 510, 950	−4. 29
850	−21, 889, 440	2, 596, 310	−8. 43
800	−51, 873, 420	4, 141, 160	−12. 53

In the above table moments are considered plus (+) when acting counterclockwise and minus (—) when acting clockwise. The eccentricity of the resultant forces is plus (+) when the resultant intersects the base upstream from the center of the base and minus (—) when the intersection is downstream from the center of the base.

An example of the calculation of the eccentricity at elevation 800 is as follows:

	Foot-pounds
Moment due to concrete weight, 3,891,750×25.47	+99, 122, 870
Moment due to vertical water load on upstream face, 15×62.5×214.5×94.47	+18, 997, 260
Moment due to vertical tail water load, ½ (32.9×62.5×47) (90.58)	−4, 376, 990
Moment due to horizontal water load upstream, ½ (62.5) $(252)^2$×⅓ (252)	−166, 698, 050

	Foot-pounds
Moment due to horizontal tail water load. $\frac{1}{2}$ (62.5) $(47)^2 \times \frac{1}{3}$ (47)	+1, 081, 490
Total moment=summation M	−51, 873, 420

	Pounds
Dead weight of concrete	3, 891, 750
Dead weight of water—upstream face 15×62.5×214.5	201, 094
Dead weight of water—tail water $\frac{1}{2}$ (32.9×62.5×47)	48, 320
Total dead weight=summation W	4, 141, 160

$$\text{Eccentricity equals } \frac{-51{,}873{,}420}{4{,}141{,}160} = -12.53 \text{ feet.}$$

TABLE 108.—*Abutment section—horizontal shear stresses—reservoir elevation 1,052.0—without earthquake effect*

Elevation	S_{su}	S_{sd}	P_u	P_d	Horizontal shear stress	
					Up-stream	Down-stream
	(1)	(1)				
1,000	+49.3	+27.1	22.6	0	0	+19.0
950	+60.6	+65.8	44.3	0	0	+46.2
900	+69.4	+108.2	66.0	0	0	+75.8
850	+77.6	+150.6	88.7	0	2.2	+105.4
800	+89.2	+194.2	109.4	20.4	4.0	+121.4

[1] From table 106.

TABLE 109.—*Abutment section—sliding factors—reservoir elevation 1,052.0—without ear.*

Elevation	Horizontal force	Weight	Uplift	Sliding factor
1,000	84, 500	264, 540	−52, 100	0. 398
950	325, 125	756, 465	−176, 565	. 561
900	722, 000	1, 510, 950	−373, 960	. 635
850	1, 275, 125	2, 596, 310	−665, 290	. 660
800	1, 915, 470	4, 141, 160	−1, 265, 110	. 666

TABLE 110.—*Abutment section—eccentricity resultant of water and dead loads and uplift—reservoir elevation 1,052.0—without earthquake effect*

Elevation	Summation M (ft.-lb.)	Summation W (lb.)	Eccentricity (ft.)
1,000	+197, 160	212, 440	+0.9
950	−2, 871, 345	579, 900	−5.0
900	−13, 842, 750	1, 136, 990	−12.2
850	−39, 419, 830	1, 931, 020	−20.4
800	−81, 052, 160	2, 876, 050	−28.2

TABLE 111.—*Abutment section—shear-friction factors of safety—reservoir elevation 1,052.0—without earthquake effect*

Elevation:	Factor of safety
1,000	34.4
950	15.9
900	10.4
850	8.1
800	7.1

Values for weight, uplift, and horizontal force are taken from table 109.

Table 112.—*Spillway section—computations of concrete weights, moments, and eccentricity—without earthquake effect*

Elevation	Area A Sq. ft.	Arm X Ft.	Moment A×X	Concrete weight A×150 Lb.	Eccentricity e
1,020–1,000: [1] A 7.67×13.45×½	51.58	20.11	1,037		
B 7.33×8.75	64.14	26.34	1,689		
C 4.70×7.33×½	17.23	25.11	433		
D 3.1416×(9.5)²(60/360)−4.5	42.72	27.40	1,170		
E 1.75×53.0	92.75	41.50	3,849		
F 6.67×6.67×½	22.24	42.45	944		
G 2.33×14.75	34.37	45.84	1,576		
H 14.75×19.75×½	145.65	53.58	7,804		
I 24.0×1.0×2/3 (App)	16.00	56.70	907		
Gates [2]	20.33	37.48	760		
Total above elevation 1,000 [3]	507.01	39.76	20,170	76,050	1.94
1,000–950: 58.77×12.9	758.13	44.39	33,650		
77.12×37.1	2,861.15	53.56	153,240		
14.0×0.3×2/3	2.80	73.50	200		
Total above elevation 950 [3]	4,129.09	50.19	207,260	619,360	9.86
950–900: 90.09×50	4,504.5	60.04	270,050		
35.00×50/2	875.0	116.75	102,156		
Total above elevation 900 [3]	9,508.6	60.94	579,470	1,426,290	16.61
900–850: 125.09×50	6,254.5	77.54	484,970		
35.00×50/2	875.0	151.76	132,790		
5.00×25/2	62.5	13.34	830		
Total above elevation 850 [3]	16,700.6	71.74	1,198,060	2,505,090	20.81
850–800: 165.09×50	8,254.5	92.54	763,871		
35.00×50/2	875.0	186.76	163,415		
10.00×50/2	250.0	6.67	1,668		
Total above elevation 800 [3]	26,080.0	81.56	2,127,010	3,912,000	23.48

[1] See table 104.
[2] Gates are assumed at 3,050 pounds per foot of spillway acting 22.48 feet downstream from the upstream face of the dam—equivalent area of concrete, i. e., 3,050/150=20.33.
[3] Each portion was divided into smaller parts and each part computed separately.

Table 113.—*Spillway section—computations of water load and moments above elevation 1,000—reservoir elevation 1,052.0—without earthquake effect*

	Area A Sq. ft.	Arm X Ft.	A×X	A×62.5 Lb.
A. 15.5×32	496.0	7.75	3,840	
B. 9.41×13	122.3	20.21	2,474	
C. 9.41×17.6/2	82.8	18.64	1,540	
D. 10×18.25/2	91.3	3.33	304	
E. 7.5×18.25	136.9	19.25	2,640	
F. 17.5×9.5/2	83.1	26.17	2,175	
	1,012.4	12.83	12,973	63,275

Arm X is the distance from the upstream face to the center of gravity of the area.

Table 114.—*Spillway section—stresses—reservoir empty—without earthquake effect*

Elevation	V Lb.	b Ft.	e Ft.	$\frac{V}{144b}$	$\frac{6e}{b}$	S_{vu} Lb./sq. in.	Stress parallel to upstream face	S_{vd} Lb./sq. in.	Stress parallel to downstream face
1,000	76,050	53.40	1.94	9.89	0.218	+12.0	+12.0	+7.7	+15.7
950	619,360	90.09	9.86	47.74	.657	+79.1	+79.1	+16.4	+24.4
900	1,426,290	125.09	16.61	79.18	.797	+142.3	+142.3	+16.1	+24.0
850	2,505,090	165.09	20.81	105.38	.756	+185.0	+192.4	+25.7	+38.3
800	3,912,000	210.09	23.48	129.31	.671	+216.1	+224.7	+42.5	+63.3

Table 115.—*Spillway section—stresses—reservoir elevation 1,052.0—without earthquake effect*

UPSTREAM FACE

Elevation	Vertical stresses (lb. per sq. in.) due to—								
	Reservoir water pressure			Tail water pressure			Concrete weight	Total vertical stress	Stress parallel to face
	Hor.	Vert.	Mom.	Hor.	Vert.	Mom.			
1,000	−15.4	+8.2	+12.8	0	0	0	+12.0	+17.6	+17.6
950	−52.1	+4.9	+10.5	0	0	0	+79.1	+42.4	+42.4
900	−93.7	+3.5	+8.4	0	0	0	+142.3	+60.5	+60.5
850	−128.3	+5.2	+13.5	0	0	0	+185.0	+75.4	+78.4
800	−155.1	+8.7	+23.2	+1.0	+1.6	−4.1	+216.1	+91.4	+95.0

DOWNSTREAM FACE

Elevation	Res. Hor.	Res. Vert.	Res. Mom.	Tail Hor.	Tail Vert.	Tail Mom.	Concrete weight	Total vertical stress	Stress parallel to face
1,000	+15.4	+8.2	−12.8	0	0	0	+7.7	+18.5	+37.7
950	+52.1	+4.9	−10.5	0	0	0	+16.4	+62.9	+93.7
900	+93.7	+3.5	−8.4	0	0	0	+16.1	+104.9	+156.3
850	+128.3	+5.2	−13.5	0	0	0	+25.7	+145.7	+217.0
800	+155.1	+8.7	−23.2	−1.0	+1.6	+4.1	+42.5	+187.8	+280.0

Table 116.—*Spillway section—eccentricity of resultant water and dead loads—reservoir elevation 1,052.0—without earthquake effect*

Elevation	Summation *M* (ft.-lb.)	Summation *W* (lb.)	Eccentricity (ft.)
1,000	−34,490	139,325	−0.25
950	−1,997,400	682,635	−2.93
900	−8,177,450	1,489,565	−5.49
850	−23,093,500	2,626,021	−8.79
800	−50,649,350	4,233,940	−11.96

Eccentricity is minus (−) as measured downstream from the center of the base investigated.

Table 117.—*Spillway section—horizontal shear stresses—reservoir elevation 1,052.0—without earthquake effect*

Elevation	S_{vu}	S_{vd}	P_u	P_d	Horizontal shear stresses	
					Upstream	Downstream
	(1)	(1)				
1,000	+17.6	+18.5	22.6	0	----------	18.2
950	+42.4	+62.9	44.3	0	----------	44.0
900	+60.5	+104.9	66.0	0	----------	73.4
850	+75.4	+145.7	88.7	0	2.6	102.0
800	+91.4	+187.8	109.4	20.4	3.6	116.9

[1] From table 115.

Table 118.—*Spillway section—sliding factors—reservoir elevation 1,052.0—without earthquake effect*

Elevation	Horizontal force	Weight	Uplift	Sliding factor
1,000	74,375	139,325	−57,850	0.913
950	315,000	682,635	−191,440	.640
900	711,875	1,489,565	−396,120	.651
850	1,265,000	2,626,021	−694,750	.655
800	1,905,340	4,223,940	−1,308,680	.653

TABLE 119.—*Spillway section—eccentricity of resultant water and dead loads and up lift—reservoir elevation 1,052.0—without earthquake effect*

Elevation	Summation M (ft.-lb.)	Summation W (lb.)	Eccentricity (ft.)
1,000	−549, 355	81, 475	−6.74
950	−4, 872, 800	491, 195	−9.92
900	−16, 436, 550	1, 093, 445	−15.0[illegible]
850	−42, 213, 000	1, 931, 271	−21.8[illegible]
800	−82, 057, 670	2, 915, 260	−28.2[illegible]

TABLE 120.—*Spillway section—shear-friction factors of safety—reservoir elevation 1,520.0—without earthquake effect*

Elevation:	*Factor of safety*
1,000	42. 07
950	17. 49
900	11. 12
850	8. 51
800	7. 35

TABLE 121.—*Abutment section—stresses—reservoir elevation 1,034.0—without earthquake effect*

UPSTREAM FACE

Elevation	Vertical stresses (lb. per sq. in.) due to—						Concrete weight	Total vertical stress	Stress parallel to face
	Reservoir water pressure			Tail water pressure					
	Hor.	Vert.	Mom.	Hor.	Vert.	Mom.			
1,000	−7.4	0	0	0	0	0	+75. 7	+68. 3	+68. 3
950	−37.3	0	0	0	0	0	+127. 3	+90. 0	+90. 0
900	−74.9	0	0	0	0	0	+178. 7	+103. 8	+103. 8
850	−108.2	+2.4	+6.9	0	0	0	+210. 6	+111. 7	+116. 0
800	−134.8	+6.3	+17.5	+0.3	+0.7	−1.9	+233. 2	+121. 3	+126. 2

DOWNSTREAM FACE

Elevation	Reservoir Hor.	Reservoir Vert.	Reservoir Mom.	Tail Hor.	Tail Vert.	Tail Mom.	Concrete weight	Total vertical stress	Stress parallel to face
1,000	+7.4	0	0	0	0	0	+0.7	+8.1	+12.1
950	+37.3	0	0	0	0	0	−.9	+36.4	+54.3
900	+74.9	0	0	0	0	0	−1.1	+73.8	+110.0
850	+108.2	+2.4	−6.9	0	0	0	+12.4	+116.1	+173.0
800	+134.8	+6.3	−17.5	+0.3	+0.7	+1.9	+33.0	+159.5	+238.0

TABLE 122.—*Abutment section—eccentricity of resultant water and dead loads—reservoir elevation 1,034.0—without earthquake effect*

Elevation	Summation M (ft.-lb.)	Summation V (lb.)	Eccentricity (ft.)
1,000	+1, 669, 870	264, 540	+6.31
950	+4, 454, 300	756, 470	+5.89
900	+5, 034, 500	1, 510, 950	+3.33
850	−1, 340, 500	2, 590, 800	−0.52
800	−18, 777, 000	4, 095, 700	−4.58

TABLE 123.—*Abutment section—horizontal shear stresses—reservoir elevation 1,034.0—without earthquake effect*

Elevation	S_{vu}	S_{vd}	P_u	P_d	Horizontal stress Upstream	Horizontal stress Downstream
	(1)	(1)				
1,000	+68.3	+8.1	14.8	0	0	+5.7
950	+90.0	+36.4	36.5	0	0	+25.4
900	+103.8	+73.8	58.1	0	0	+51.7
850	+111.7	+116.1	79.9	0	−6.4	+81.3
800	+125.1	+155.7	101.6	13.0	−4.7	+100.0

[1] From table 121.

TABLE 124.—*Abutment section—sliding factors—reservoir elevation 1,034.0—without earthquake effect*

Elevation	Horizontal force	Weight	Uplift	Sliding factor
1,000	36,125	264,540	34,060	0.157
950	220,500	756,470	145,380	.361
900	561,125	1,510,950	329,615	.475
850	1,058,000	2,590,800	605,900	.533
800	1,683,000	4,095,700	1,116,800	.565

TABLE 125.—*Abutment section—shear-friction factors of safety—reservoir elevation 1,034.0—without earthquake effect*

Elevation:	*Factor of safety*
1,000	80.8
950	23.5
900	13.5
850	9.9
800	8.1

TABLE 126.—*Abutment section—computations of moments and eccentricity—with earthquake effect*

Elevation	Area A	Arm X	Moment	M_E	M_V+M_E	Eccentricity
1,060–1,040.13	397.4	50.07	19,898			
1,040.13–1,000	802.6	20.07	16,108			
	563.6	13.38	7,541			
Total above elevation 1,000	1,763.6	24.69	43,547	+653,150	+2,732,450	+10.33
	1,763.6	74.69	131,723			
1,000–950	2,404.5	25.00	60,113			
	875.0	16.67	14,586			
Total above elevation 950	5,043.1	40.93	206,422	+3,096,200	+13,724,500	+18.14
	5,043.1	90.93	458,570			
950–900	4,154.5	25.00	103,863			
	875.0	16.67	14,586			
Total above elevation 900	10,073	57.28	577,019	+8,654,700	+38,752,700	+25.65
	10,073.0	107.28	1,080,630			
900–850	5,904.5	25.00	147,610			
	875.0	16.67	14,586			
	62.5	8.33	520			
Total above elevation 850	16,915	73.51	1,243,346	+18,651,000	+78,099,000	+30.78
	16,915	123.51	2,089,170			
850–800	7,905	25.00	197,630			
	875	16.67	14,590			
	250	16.67	4,170			
Total above elevation 800	25,945	88.95	2,305,560	+34,617,000	+133,740,000	+34.37

Area A from table 104.
Arm X is vertical distance from the center of gravity of the area to the base being investigated.
$M_E = V$ times arm X.
Eccentricity $= (M_V+M_E)/V$.
$M_V = Ve$ (Values of V and e from table 105).

TABLE 127.—*Abutment section—stresses—reservoir empty—with earthquake effect*

Elevation	V Lb.	b Ft.	e_E Ft.	$\frac{V}{144b}$	$\frac{6e_E}{b}$	S_{uE} Lb./sq. in.	Stress parallel to upstream face S_{puE}	S_{dE} Lb./sq. in.	Stress parallel to downstream face S_{pdE}
1,000	264,540	48.09	10.33	38.2	1.289	+87.4	+87.4	−11.0	−16.4
950	756,465	83.09	18.14	63.2	1.310	+146.0	+146.0	−19.6	−29.2
900	1,510,950	118.09	25.65	88.9	1.303	+204.7	+204.7	−26.9	−40.1
850	2,537,250	158.09	30.78	111.5	1.168	+241.7	+251.4	−18.7	−27.9
800	3,891,750	203.09	34.37	133.1	1.015	+268.2	+278.9	−2.0	−3.0

TABLE 128.—*Abutment section—shear stresses on horizontal plane—reservoir empty—with and without earthquake effect*

Elevation	Upstream face		Downstream face	
	Without earthquake	With earthquake	Without earthquake	With earthquake
1,000	0	0	+0.5	−7.7
950	0	0	−.6	−13.7
900	0	0	−.8	−18.8
850	−42.1	−48.3	+8.7	−13.1
800	−46.6	−53.6	+23.1	−1.4

TABLE 129.—*Abutment section—stresses—reservoir elevation 1,034.0—with earthquake effect*

Elevation	$\frac{V}{144b}$	M	M_E	M_{wE}	Summation M	S_{uE}	Stress parallel to upstream face	S_{dE}	Stress parallel to downstream face
1,000	38.2	+1,669,870	−653,150	−143,180	+873,540	+53.9	+53.9	+22.5	+33.5
950	63.2	+4,454,300	−3,096,200	−1,371,400	−13,300	+63.1	+63.1	+63.3	+94.3
900	88.9	+5,034,500	−8,654,700	−4,413,400	−8,033,600	+64.9	+64.9	+112.9	+168.3
850	113.8	−1,340,500	−18,651,000	−9,750,800	−29,742,300	+64.2	+66.8	+163.4	+243.5
800	140.0	−18,777,000	−34,617,000	−17,821,000	−71,215,000	+68.1	+70.8	+211.9	+315.8

TABLE 130.—*Abutment section—eccentricity of resultant water, dead, and earthquake loads—reservoir elevation 1,034.0*

Elevation	Summation M (ft.-lb.)	Summation V (lb.)	Eccentricity (ft.)
1,000	+873,540	264,540	+3.30
950	−13,300	756,470	−.02
900	−8,033,600	1,510,950	−5.32
850	−29,742,300	2,590,800	−11.48
800	−71,215,000	4,095,700	−17.39

TABLE 131.—*Abutment section—horizontal shear stresses—reservoir elevation 1,034.0—with earthquake effect*

Elevation	S_{wuE}	S_{xzE}	P_{wE}	P_{dE}	Horizontal shear stresses	
					Up-stream	Down-stream
	(1)	(1)				
1,000	53.9	22.5	18.0	0	0	+15.7
950	63.1	63.3	41.6	0	0	+44.3
900	64.9	112.9	64.5	0	0	+79.0
850	64.2	163.4	87.4	0	+4.6	+114.4
800	68.1	211.9	109.8	13.0	+8.3	+139.3

[1] From table 129.

TABLE 132.—*Abutment section—horizontal forces—reservoir elevation 1,034.0—with earthquake effect*

Elevation	Horizontal water	G	P_{wE}	Total
1,000	36,125	26,454	10,521	73,100
950	220,500	75,647	40,853	337,000
900	561,125	151,095	82,310	794,530
850	1,058,000	253,725	132,450	1,444,175
800	1,683,000	389,175	[1] 193,000	2,265,175

[1] Includes 3,050 pounds for P_{wE} due to tail water inertia.

G=Horizontal inertia force of concrete=KV.

P_{wE}=Horizontal inertia force of water=$(2/3)CKh\sqrt{zh}$.

TABLE 133.—*Abutment section—sliding factors—reservoir elevation 1,034.0—with earthquake effect*

Elevation	Horizontal force	Weight	Uplift	Sliding factor
1,000	73,100	264,540	34,060	0.317
950	337,000	756,470	145,380	.551
900	794,530	1,510,950	329,615	.673
850	1,444,175	2,590,800	605,900	.728
800	2,265,175	4,095,700	1,116,800	.760

TABLE 134.—*Abutment section—shear-friction factors of safety—reservoir elevation 1,034.0—with earthquake effect*

Elevation:	*Factor of safety*
1,000	39.9
950	15.4
900	9.5
850	7.2
800	6.0

TABLE 135.—*Spillway section—computations of water load and moments above elevation 1,000—reservoir elevation 1,034.0—without earthquake effect*

	Area A Sq.ft.	Arm X Ft.	A×X	A×62.5 Lb.
14.0×15.5	217.0	7.75	1,682	
9.41×14.0/2	65.8	18.64	1,228	
10×18.25/2	91.3	3.33	304	
7.5×18.25	136.9	19.25	2,635	
18.25×9.5/2	86.7	26.17	2,269	
Total	597.7	13.58	8,118	37,400

TABLE 136.—*Spillway section—stresses—reservoir elevation 1,034.0—without earthquake effect*

UPSTREAM FACE

Elevation	Vertical stresses (lb./sq. in.) due to— Reservoir water pressure: Hor.	Reservoir water pressure: Vert.	Reservoir water pressure: Mom.	Tail water pressure: Hor.	Tail water pressure: Vert.	Tail water pressure: Mom.	Vertical stresses due to concrete weight[1]	Total vertical stress	Stress parallel to face
1,000	−6.0	+4.9	+7.2				+12.0	+18.1	+18.1
950	−31.7	+2.9	+6.1				+79.1	+56.4	+56.4
900	−66.7	+2.1	+4.9				+142.3	+82.6	+82.6
850	−99.2	+3.9	+10.3				+185.0	+100.0	+104.0
800	−126.0	+7.3	+20.8	+0.3	+0.7	−1.8	+216.1	+117.4	+122.0

DOWNSTREAM FACE

Elevation	Reservoir water pressure: Hor.	Reservoir water pressure: Vert.	Reservoir water pressure: Mom.	Tail water pressure: Hor.	Tail water pressure: Vert.	Tail water pressure: Mom.	Vertical stresses due to concrete weight[1]	Total vertical stress	Stress parallel to face
1,000	+6.0	+4.9	−7.2				+7.7	+11.4	+23.2
950	+31.7	+2.9	−6.1				+16.4	+44.9	+67.0
900	+66.7	+2.1	−4.9				+16.1	+80.0	+119.2
850	+99.2	+3.9	−10.3				+25.7	+118.5	+176.5
800	+126.0	+7.3	−20.8	−0.3	+0.7	+1.8	+42.5	+157.2	+235.0

[1] See table 114.

TABLE 137.—*Spillway section—eccentricity of resultant water and dead loads—reservoir elevation 1,034.0—without earthquake effect*

Elevation	Summation M (ft.-lb.)	Summation V (lb.)	Eccentricity (ft.)
1,000	+230,500	113,450	+2.03
950	+1,118,000	656,760	+1.70
900	+430,000	1,463,690	+.29
850	−5,910,000	2,596,090	−2.28
800	−21,761,000	4,153,100	−5.25

TABLE 138.—*Spillway section—horizontal shear stresses—reservoir elevation 1,034.0—without earthquake effect*

Elevation	S_{su} [1]	S_{sd} [1]	P_u	P_d	Horizontal shear stress: Upstream	Horizontal shear stress: Downstream
1,000	18.1	11.4	14.8	0	0	+11.0
950	56.4	44.9	36.5	0	0	+31.6
900	82.6	80.0	58.1	0	0	+56.0
850	100.0	118.5	79.9	0	−4.0	+83.0
800	117.4	157.2	101.3	13.0	−3.3	+101.0

[1] From table 136.

TABLE 139.—*Spillway section—sliding factors—reservoir elevation 1,034.0—without earthquake effect*

Elevation	Horizontal force	Weight	Uplift	Sliding factor
1,000	36,125	113,450	37,830	0.477
950	220,500	656,760	157,660	.441
900	561,125	1,463,690	349,260	.504
850	1,058,000	2,596,090	632,900	.539
800	1,683,000	4,153,100	1,155,600	.562

TABLE 140.—*Spillway section—eccentricity of resultant water and dead loads and uplift—reservoir elevation 1,034.0—without earthquake effect*

Elevation	Summation M (ft.-lb.)	Summation loads (lb.)	Eccentricity (ft.)
1,000	−106,000	75,620	−1.42
950	−1,247,000	499,100	−2.50
900	−6,850,000	1,114,430	−6.15
850	−23,310,000	1,963,190	−11.85
800	−55,359,800	2,997,500	−18.42

TABLE 141.—*Spillway section—shear-friction factors of safety—reservoir elevation 1,034.0—without earthquake effect*

Elevation:	*Factor of safety*
1,000	86.5
950	25.0
900	14.1
850	10.2
800	8.3

TABLE 142.—*Spillway section—computations of moments and eccentricity—with earthquake effect*

Elevation	Area A	Arm X	Moment	M_E	M_V+M_E	Eccentricity
1,020–1,000	A 51.58	6.23	321			
	B 64.14	6.12	393			
	C 17.23	12.07	208			
	D 42.72	16.00	684			
	E 92.75	.87	81			
	F 22.24	3.97	88			
	G 34.37	9.12	313			
	H 145.65	6.66	970			
	I 16.00	9.50	152			
Gates [1]	20.33	24.00	488			
Total above elevation 1,000	507.01	7.30	3,698	+55,500	+203,000	+2.67
1,000–950	507.01	57.30	29,050			
	758.13	43.35	32,865			
	2,861.15	17.50	50,080			
	2.80	44.50	125			
Total above elevation 950	4,129.09	27.20	112,120	+1,682,000	+7,782,000	+12.57
950–900	4,129.09	77.20	319,000			
	4,504.5	25.00	112,600			
	875.0	16.66	14,580			
Total above elevation 900	9,508.6	46.84	446,180	+6,700,000	+30,400,000	+21.29
900–850	9,508.6	96.84	920,800			
	6,254.5	25.00	156,360			
	875.0	16.66	14,580			
	62.5	8.33	520			
Total above elevation 850	16,700.6	65.40	1,092,260	+16,383,300	+68,514,000	+27.35
850–800	16,700.6	115.40	1,927,400			
	8,254.5	25.00	206,400			
	875.0	16.66	14,600			
	250.0	16.66	4,200			
Total above elevation 800	26,080.0	82.54	2,152,600	+32,289,000	+124,143,000	+31.73

[1] Gates are assumed at 3,050 pounds per foot of spillway. Equivalent area of concrete, i. e., 3,050/150=20.33.

Area A from table 112.
Arm X is vertical distance from the center of gravity of the area to the base being investigated.
$M_E=V$ times Arm X.
Eccentricity$=(M_V+M_E)/V$.
$M_V=Ve$ (values of V and e from table 114).

TABLE 143.—*Spillway section—stresses—reservoir empty—with earthquake effect*

Elevation	V Lb.	b Ft.	e_E Ft.	$\frac{V}{144b}$	$\frac{6e_E}{b}$	S_{vE} Lb./sq. in.	Stress parallel to up-stream face	S_{vdE} Lb./sq. in.	Stress parallel to down-stream face
1,000	76,050	53.40	2.67	9.89	0.30	+12.8	+12.8	+7.0	+14.3
950	619,360	90.09	12.57	47.74	.837	+87.7	+87.7	+7.8	+11.6
900	1,426,390	125.09	21.29	79.18	1.021	+160.0	+160.0	−1.6	−2.4
850	2,505,090	165.09	27.35	105.38	.994	+210.1	+218.5	+0.6	+0.9
800	3,912,000	210.09	31.73	129.31	.906	+246.5	+256.4	+12.2	+18.2

TABLE 144.—*Spillway section—shear stresses on horizontal plane—reservoir empty—with and without earthquake effect*

Elevation	Upstream face		Downstream face	
	Without earthquake	With earthquake	Without earthquake	With earthquake
1,000	0	0	+7.5	+6.8
950	0	0	+11.5	+5.4
900	0	0	+11.3	−1.1
850	−37.0	−42.0	+18.0	+.4
800	−43.2	−49.3	+29.8	+8.5

TABLE 145.—*Spillway section—stresses—reservoir elevation 1,034.0—with earthquake effect*

Elevation	$\frac{W}{144b}$	M	M_E	M_{wE}	Summation M	S_{vuE}	Stress parallel to up-stream face	S_{vdE}	Stress parallel to down-stream face
1,000	14.75	+230,500	−55,500	−143,020	+31,980	+15.2	+15.2	+14.3	+29.1
950	50.7	+1,118,000	−1,682,000	−1,371,400	−1,935,400	+40.7	+40.7	+60.8	+90.6
900	81.1	+430,000	−6,700,000	−4,413,400	−10,683,400	+52.7	+52.7	+109.7	+163.5
850	109.1	−5,910,000	−16,383,300	−9,750,800	−32,043,800	+60.0	+62.4	+158.5	+236.2
800	137.2	−21,761,000	−32,289,000	−17,821,000	−71,871,000	+69.3	+72.0	+205.0	+305.5

TABLE 146.—*Spillway section—eccentricity of resultant water, dead, and earthquake loads—reservoir elevation 1,034.0*

Elevation	Summation M (ft.-lb.)	Summation loads (lb.)	Eccentricity (ft.)
1,000	+31,980	113,450	+0.28
950	−1,935,400	656,760	−2.95
900	−10,683,400	1,463,690	−7.3
850	−32,043,800	2,596,090	−12.33
800	−71,871,000	4,153,100	−17.3

TABLE 147.—*Spillway section—horizontal shear stresses—reservoir elevation 1,034.0—with earthquake effect*

Elevation	S_{uE}	S_{dE}	P_{uE}	P_{dE}	Horizontal shear stress	
					Upstream	Down-stream
	(1)	(1)				
1,000	+15.2	+14.3	18.0	0	0	+14.0
950	+40.7	+60.8	41.6	0	0	+42.5
900	+52.7	+109.7	64.5	0	0	+76.8
850	+60.0	+158.5	87.4	0	+5.5	+111.0
800	+68.3	+205.0	109.8	13.0	+8.3	+134.5

[1] From table 145.

TABLE 148.—*Spillway section—sliding factors—reservoir elevation 1,034.0—with earth quake effect*

Elevation	Horizontal force	Weight	Uplift	Sliding factor
1,000	54,251	113,450	37,830	0.717
950	323,300	656,760	157,660	.648
900	786,060	1,463,690	349,260	.706
850	1,441,000	2,596,090	632,900	.735
800	2,267,200	4,153,100	1,155,600	.757

TABLE 149.—*Spillway section—shear-friction factors of safety—reservoir elevation 1,034.0—with earthquake effect*

Elevation:	*Factor of safety*
1,000	57.6
950	17.0
900	10.1
850	7.5
800	6.2

FOUNDATION STABILITY

The foundation stability was checked, based on the assumption that if sliding were to occur the plane of weakness would be on a plane in the rock.

Sample computations for stability analysis of the abutment section, based on condition 3 (see page 74), are as follows:

VERTICAL PRESSURES

	Pounds
Weight of concrete above elevation 800.0. (See table 104—Check stability analysis)	3,891,800
Weight of reservoir water. $\frac{252+177}{2}\times 15\times 62.5$	201,100
Weight of tail water acting on the dam, $\frac{47}{2}\times 32.9\times 62.5$	48,300
Total weight of concrete and water	4,141,200
Weight of rock triangle under the dam, $\frac{203.1\times 11.6}{2}\times 160$	188,500

	Pounds
Weight of water[1] acting on the triangle, $0.50 \times 62.5 \times \frac{47+58.6}{2} \times 17.8$	29,400
Total weight of rock triangle and water acting on triangle	217,900
Total weight above plane of analysis	4,359,100
Less uplift on 65 percent of nonintact portions (vertical component), $0.65 \times 62.5 \times \frac{252+58.6}{2} \times 220.9$	−1,393,700
Net vertical pressure	2,965,400

[1] One-half the hydrostatic pressure due to tail water acting on the rock triangle is based on the assumption that the proportion of intact rock in the plane of shearing in the toe rock is considered as 50 percent.

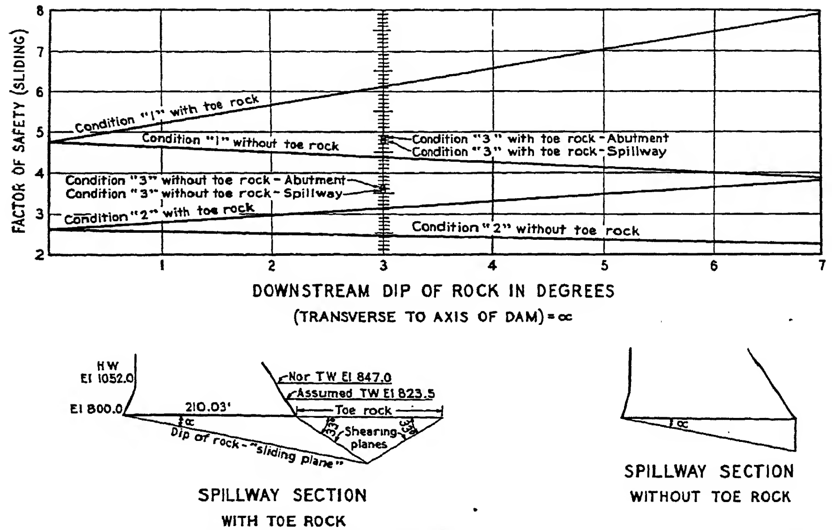


FIGURE 326.—*Factor of safety against sliding for conditions 1 and 2 with and without toe rock.*

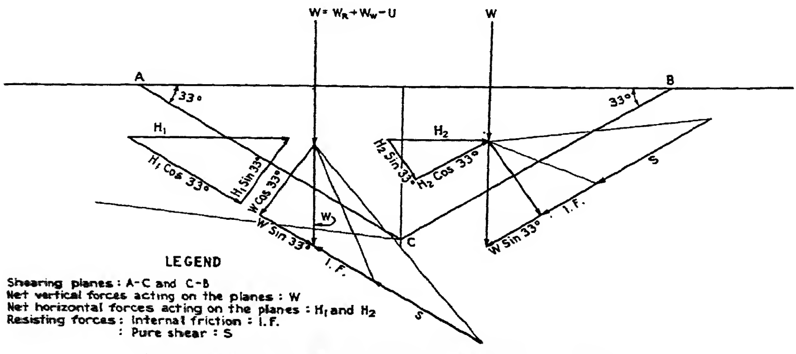


FIGURE 327.—*Diagram of forces for resisting toe rock.*

Horizontal Pressures

	Pounds
Water pressure upstream face (above elevation 800.0), $0.50\times62.5\times(252)^2$	1,984,500
Water pressure downstream face (above elevation 800.0), $0.50\times62.5\times(47)^2$	−69,000
Total horizontal water pressures	1,915,500
Component of uplift, 1,393,700×Tan 3°	73,000
Total horizontal pressure above elevation 800.0	1,988,500
Less tail water pressure below elevation 800.0, $0.50\times62.5\times\frac{47.0\times58.6}{2}\times11.6$	−19,100
Net horizontal pressure	1,969,400

Pressure Normal to Plane (Inclined 3°)

Component of vertical pressure, 2,965,400×Cos 3°	2,961,300
Less component of horizontal pressure, 1,969,400×Sin 3°	−103,100
Net pressure normal to plane	2,858,200

Resistance to Sliding Along Base (Intact Area 35 Percent)

Resistance to pure shear, $\frac{220.9}{\text{Cos }3^\circ}\times144\times0.35\times500$	5,574,300
Internal friction, 2,858,200×0.75	2,143,700
Total resistance in gross shear under the dam	7,718,000
Less component of pressure normal to 3° plane, 2,858,200×Sin 3°	−149,600
Net resistance along base	7,568,400
Net horizontal component of resistance along base, 7,568,400×Cos 3°	7,558,000

Resistance of Toe Rock

(See fig. 326)

Total weight of rock triangle ($2W_R$), 35.6×11.6×160/2	33,000
Weight of water on rock triangle ($2W_W$), 62.5×47×35.6	104,600
Less uplift ($2U$), $0.50\times62.5\times\left(\frac{47+58.6}{2}\right)\times35.6$	−58,700
Total vertical weight ($2W$)	78,900
or W=78,900/2	39,500
Component normal to plane, 39,500 Cos 33°	33,200
Component parallel to plane, 39,500 Sin 33°	21,500
Pure shear (S) along the plane, 0.50×21.2×144×600	915,900

Internal friction=(33,200−H_1 Sin 33°)×0.75 and (33,200+H_2 Sin 33°)×0.75

Therefore,

$$H_1 \text{ Cos } 33^\circ = 915{,}900 - 21{,}500 + (33{,}200 - H_1 \text{ Sin } 33^\circ)\ 0.75$$

or

$$H_1 = \frac{894{,}400 + 24{,}900}{\text{Cos } 33^\circ + 0.75 \text{ Sin } 33^\circ} = \frac{919{,}300}{1.24715} = 737{,}100 \text{ lb.}$$

and

$$H_2 \text{ Cos } 33° = 915{,}900 + 21{,}500 + (33{,}200 + H_2 \text{ Sin } 33°)\ 0.75$$

or

$$H_2 = \frac{937{,}400 + 24{,}900}{\text{Cos } 33° - 0.75 \text{ Sin } 33°} = \frac{962{,}400}{0.43019} = 2{,}237{,}000 \text{ lb.}$$

The total resisting force due to rock would, therefore, be $H_1 + H_2$ but since there is uncertainty as to the action of H_1 (H_1 would be zero if it were to be considered that the rock along the plane would not take tension), this force will be neglected and only H_2 considered.

	Pounds
Net resisting force therefore equals H_2	2, 237, 000
Plus net horizontal component of resistance along the base	7, 558, 000
Total	9, 795, 000

FACTORS OF SAFETY (SLIDING)

With toe rock

$$\frac{\text{Forces resisting sliding}}{\text{Forces tending to cause sliding}} = \frac{9{,}795{,}000}{1{,}969{,}400} = 4.97$$

Without toe rock

$$\text{Forces resisting sliding, } 7{,}558{,}000 - \frac{221.2 - 203.4}{221.2} \times 5{,}574{,}300 = 7{,}117{,}900 \text{ lb.}$$

$$\frac{\text{Forces resisting sliding}}{\text{Forces tending to cause sliding}} = \frac{7{,}117{,}900}{1{,}969{,}400} = 3.61$$

Sample computations for stability analysis of the spillway section, based on Assumption 3, are as follows:

VERTICAL PRESSURES

	Pounds
Weight of concrete above elevation 800 (including gates) (see table 110; check stability analysis)	3, 912, 000
Weight of reservoir water:	
$\frac{252+177}{2} \times 15 \times 62.5$	201, 100
Plus (see table 113; check stability analysis)	63, 300
Weight of tail water on dam (tail water at elevation 823.5):	
$62.5 \times \frac{(23.5)^2\ (0.7)}{2}$	12, 100
Weight of water acting on rock triangle:	
$0.5 \times 62.5 \times \frac{47+59}{2} \times 18.5$	30, 600
Total water load	307, 100
Total concrete and water load	4, 219, 100
Weight of rock triangle under dam:	
$\frac{228.6 \times 12}{2} \times 160$	219, 500
Total weight above plane[1]	4, 438, 600
Uplift of 65 percent of "Nonintact portions" (vertical component):	
$0.65 \times 62.5 \times \frac{252+59}{2} \times 228.6$	−1, 444, 100
Net vertical pressure on plane	2, 994, 500

[1] Neglecting weight of tail water acting on the rock triangle under the dam.

Horizontal Pressures

	Pounds
Reservoir water pressure, $0.5 \times 62.5 \times (252^2 - 18^2)$	1,974,400
Component of uplift, $1{,}444{,}100 \times \text{Tan } 3°$	75,700
Net horizontal pressure [1]	2,050,100

[1] Neglecting tail water.

Pressure Normal to Plane (Inclined 3°)

Component of vertical pressure, $2{,}994{,}500 \times \text{Cos } 3°$	2,990,400
Less component of vertical pressure, $2{,}050{,}100 \times \text{Sin } 3°$	107,300
Net pressure normal to plane	2,883,100

Resistance to Sliding Along Base (Intact Area 35 Percent)

Resistance to pure shear, $\dfrac{228.6}{\text{Cos } 3°} \times 144 \times 0.35 \times 500$	5,768,500
Internal friction, $2{,}883{,}100 \times 0.75$	2,162,300
Total resistance in gross shear under dam	7,930,800
Less component of vertical force, $2{,}883{,}100 \times \text{Sin } 3°$	−150,900
Net resistance along base	7,779,900
Net horizontal resistance along base, $7{,}779{,}900 \times \text{Cos } 3°$	7,769,200

Resistance of Toe Rock

(See fig. 327)

Total weight of rock triangle, $37.0 \times 12.0 \times 0.5 \times 160$	35,500
Weight of water acting on rock triangle (assumed at elevation 823.5), $62.5 \times 23.5 \times 37.0$	54,300
Less uplift, $0.50 \times 62.5 \times \dfrac{23.5+35.5}{2} \times 37.0$	−34,100
Total vertical weight	55,700
or $W = 55{,}700/2$	27,900
Component normal to plane, 27,900 Cos 33°	23,400
Component parallel to shear plane, 27,900 Sin 33°	15,200
Pure Shear (S) along plane, $0.5 \times 22.0 \times 144 \times 600$	950,400

Internal friction, $(23{,}400 - H_1 \text{ Sin } 33°) \times 0.75$ plus $(23{,}400 + H_2 \text{ Sin } 33°) \times 0.75$
Therefore

$$H_1 \text{ Cos } 33° = 950{,}400 - 15{,}200 + (23{,}400 - H_1 \text{ Sin } 33°)\ 0.75$$

or

$$H_1 = \frac{935{,}200 + 17{,}500}{\text{Cos } 33° + 0.75 \text{ Sin } 33°} = \frac{952{,}700}{1.24715} = 763{,}900$$

and

$$H_2 \text{ Cos } 33° = 950{,}400 + 15{,}200 + (23{,}400 + H_2 \text{ Sin } 33°)\ 0.75$$

or

$$H_2 = \frac{965{,}600 + 17{,}500}{\text{Cos } 33° - 0.75 \text{ Sin } 33°} = \frac{983{,}100}{0.43019} = 2{,}285{,}300$$

The total resisting force due to rock would therefore, be $H_1 + H_2$, but since there is uncertainty as to the action of H_1, (H_1 would be zero if it were to be considered that the rock along the plane A–C would not take tension), this force will be neglected and only force H_2 considered.

	Pounds
Net resisting force therefore equals H_2	2,285,300
Plus net horizontal component of resistance along the base	7,769,200
Total	10,054,500

FACTORS OF SAFETY (SLIDING)

With toe rock

$$\frac{\text{Forces resisting sliding}}{\text{Forces tending to cause sliding}}=\frac{10,054,500}{2,050,100}=4.90$$

Without toe rock

Forces resisting sliding

$$7,769,200-\frac{228.6-210.1}{228.6}\times 5,768,500=7,302,400$$

$$\frac{\text{Forces resisting sliding}}{\text{Forces tending to cause sliding}}=\frac{7,302,400}{2,050,100}=3.56$$

Appendix D

MODEL STUDIES

In the design of a dam, many features are not determinable mathematically, and where this is the case, or where a mathematical analysis must be tested for accuracy, model studies may be used. Model studies were made of Norris Dam, first, to determine the hydraulic characteristics and architectural effect of the various proposed designs of the spillway, apron, drum gates, and outlet conduits; second, to check the mathematical stress analyses for the dam; and third, to determine some mechanical and hydraulic characteristics of the tractor gates used to close the power intakes.

All of the model tests were made by the United States Bureau of Reclamation for the Tennessee Valley Authority. The Bureau's reports on the various model studies have been summarized in this appendix. The results, conclusions, and a brief description of the equipment, testing procedures, and tests which led up to the final design are given in every case. For further details the appropriate report of the Bureau of Reclamation should be consulted.

HYDRAULIC MODEL STUDIES

Hydraulic models of Norris Dam were constructed and tested as a part of the studies made of the hydraulic characteristics of the various proposed designs of the dam. The use of a model permitted the study and solution of many problems which could not be solved by mathematical or other methods. The studies[1] were made by the United States Bureau of Reclamation.

The model investigations were divided into three main parts: spillway studies, studies of auxiliary passages, and architectural studies. The spillway studies consisted of the determination of good stilling pool characteristics, the calibration of the drum gates, the determination of pressures on the drum gates, the measurement of velocities on the apron, the determination of the size of the training walls and the pressures exerted on them, and the development of a drum gate operating schedule. The studies on the auxiliary passages included tests on the outlet conduits to determine a favorable hydraulic design and an operating schedule, tests on needle valve outlets through the powerhouse, and tests to determine the hydraulic characteristics of the powerhouse water discharge. The architectural studies were made of the piers and the spillway highway bridge.

The floodwaters, with a probable maximum discharge of 240,000 cubic feet per second and a fall of 246 feet could create a maximum of 6,700,000 horsepower. If these forces were not made harmless, great danger to the dam foundation would result. Hence it was especially important that the design of the dam, and particularly the stilling pool, be structurally and hydraulically sound.

MODEL

The model was built to a scale of 1:72. The relations of the dimensions and quantities of the model and prototype based on the laws of hydraulic similitude, considering the forces of gravity and inertia only, are shown in table 150.

The model was constructed as shown in figure 330, using steel structural shapes, sheet metal, concrete, and a small amount of lumber. The drum gates were operated mechanically. Downstream from the dam the topographic features were reproduced by pouring a 1-inch concrete slab to rock elevation and covering this slab with sand and fine gravel to form the overburden.

[1] Thomas, C. W., Hydraulic Model Experiments for the Design of the Norris Dam, U. S. Bureau of Reclamation Technical Memorandum No. 406, October 15, 1934.

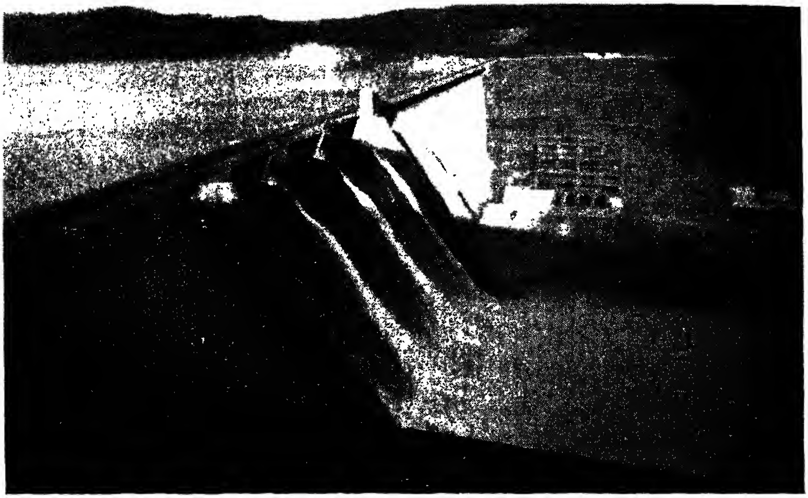


FIGURE 328.—*Prototype—In operation in February 1937.*

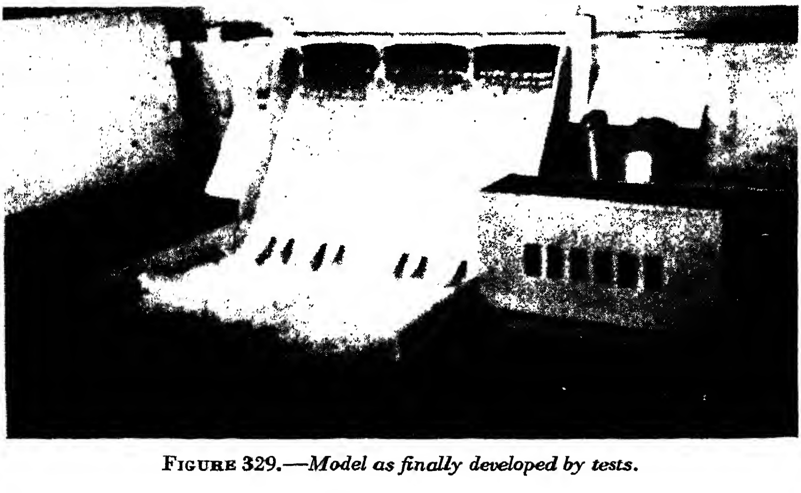


FIGURE 329.—*Model as finally developed by tests.*

TABLE 150.—*Scale relationships*

Quantity	Formula[1]	Value
Head	N	72
Other linear dimensions	N	72
Velocity	$\sqrt{N}$	8.48
Area	N^2	5,184
Discharge	$N^{5/2}$	43,985
Time	$\sqrt{N}$	8.48

[1] N denotes the scale ratio.

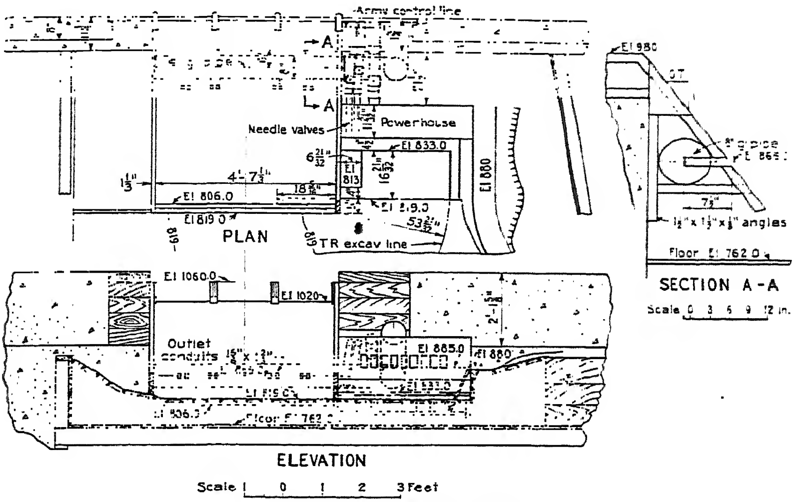


FIGURE 330.—*Model design.*

Spillway studies

The proper design of the stilling pool is of vital importance to the safety of a high dam due to the large amount of energy that must be dissipated at its base. Only in the past few years has a great deal of study been given to determining the best methods of protecting the stream beds below dams from scour; however, many different solutions have resulted from a great variety of conditions.

Preliminary studies.

A determination by means of model testing for each individual case has rapidly grown into use, even though the principles governing the design of scour protection have been worked out and the types of protection applicable to conditions for various types of dam sites have been classified.[2] Individual testing is necessary to determine which of the several forms is applicable to the conditions at a given dam site. The classification of principles, however, does narrow the necessary field of study and insures that unsatisfactory conditions are not overlooked.

Briefly, the type of scour protection depends upon the relationship, throughout the entire range of discharge, of the tail water elevation below the dam to

[2] Lane, E. W., and Bingham, W. F., Protection Against Scour Below Overfall Dams, United States Bureau of Reclamation Technical Memorandum No. 323, January 23, 1933

the tail water elevation necessary to cause a hydraulic jump to form on a level apron at the elevation of the stream bed, or in other words, upon the relative position of the tail water rating curve and a jump height curve for an apron at stream bed level. This is true whether or not the hydraulic jump is to be used for scour protection.

The ideal condition would be to have a tail water at such a height above the river bed for each discharge that it would form a perfect hydraulic jump for the depth and velocity which would occur in the overfalling stream at the toe of the dam for that discharge. The height of the tail water, however, is controlled by the conditions in the stream channel downstream from the dam, and this ideal condition is never exactly attained. Other than this perfect condition, there are four classes into which those various tail water rating curve jump height curve relationships can fall. These are shown in figure 331.

The conditions at Norris are very favorable to the protection from scour by simple means. As will be seen in figure 332, the jump height curve for the apron at stream bed level is considerably above the tail water rating curve for all discharges. Norris, therefore, falls into class II. (See fig. 331.)

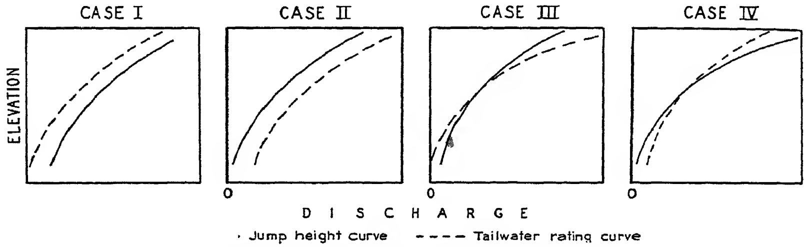


Figure 331.—*Relationships of jump height and tail water rating curves.*

For this condition the possible solutions are (1) sloping the bucket to throw the overfalling stream far from the base of the dam, (2) raising the tail water level by a secondary dam, (3) deepening the pool below the dam, and (4) installing various forms of baffles.

The first mentioned of these solutions with the stratified rock bottom and the very high heads acting at Norris would produce severe scour in the river bed downstream from the dam and pile up debris in the stream channel below the dam which would probably obstruct the flow from the powerhouse. Since a very good solution is possible by another one of the four methods, this method, which would have undesirable results, was not given serious consideration.

Satisfactory protection from the hydraulic standpoint could undoubtedly be secured by means of the tail water dam mentioned as the second possible solution. The secondary dam, when used, must be high enough to produce the required tail water level for all discharges. Figure 332 shows that a weir with crest at elevation 859.6 would be required to hold the water to the height necessary for a discharge of 250,000 cubic feet per second. Such a dam would be about 40 feet high and would require training walls 70 feet high along both sides of the pool. Moreover, the pool would have to be of considerable length; 4½ to 5 times the height of the jump is required to produce satisfactory conditions where the height of the secondary dam is an appreciable part of the pool depth. Some protection against scour would also be required below the secondary dam, but since (as shown on figure 332) the natural tail water curve is nearly high enough to produce a jump below the secondary dam on an apron at stream bed level, little in addition to the horizontal apron below the secondary weir would be required. The cost of the 40-foot tail water dam with its apron and 70-foot retaining walls, however, is so great that this form of protection, although undoubtedly satisfactory from the hydraulic standpoint, would be entirely too expensive.

The use of baffles of the ordinary forms to dissipate the energy of the water passing over the dam was not favored because of the unprecedented velocities which would act on them and the difficulty of building them strong enough to resist the blows to which they might be subjected by logs. The large difference

in height between the tail water rating curve and the curve for height of jump on an apron at stream bed level indicates that the energy which would have to be dissipated by any form of baffle would be large and therefore would be subjected to large impacts. Several forms of baffles were tested, however, in an attempt to shorten the length of depressed apron downstream.

The best form of protection is secured by deepening the pool by excavation, placing the pool bottom at elevation 806, or about 13 feet below the stream bed level. For this condition the depth of pool is sufficient to form a jump at all flows. The close agreement between the jump height curve and the tail water curve indicates that a nearly perfect hydraulic jump would be formed in which the energy would be efficiently dissipated.

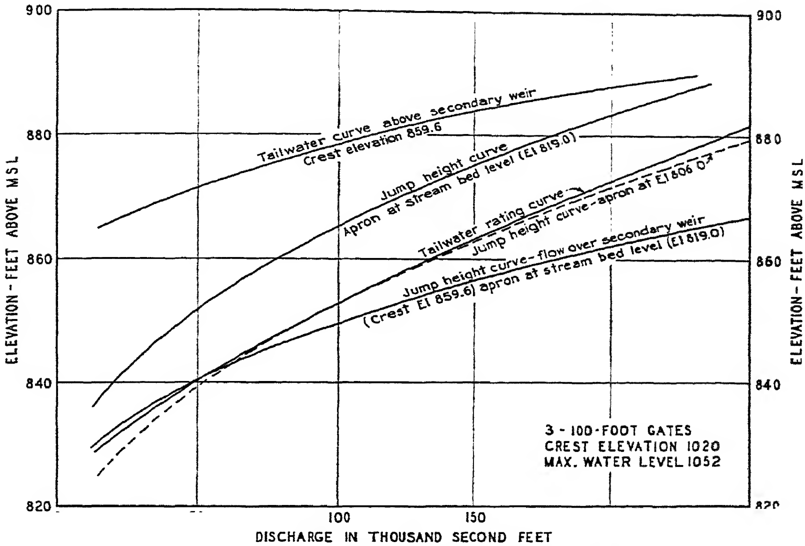


FIGURE 332.—*Jump height and tail water rating curves for Norris.*

The action of the pool as shown on figure 332 is based on the assumption that the drum gates are kept at the same elevation. For the lower discharges this is not so important since the pool is deeper than necessary for this condition and some unbalancing of the gate discharge would be permissible.

From this preliminary analysis of the conditions, the range of model tests was narrowed appreciably.

Tests and results.

The problem of designing the stilling pool was complicated throughout the tests by the necessity of meeting the requirements of the three following conditions: (1) outlet conduits discharging singly or in combination, (2) spillway discharging, and (3) outlet conduits and spillway, both discharging. In many instances a set-up would fulfill one or more of these requirements but would be objectionable for the remainder.

Throughout the series of tests on various aprons, a representative number of runs were made on each test. Model flows representing prototype flows of 20,000, 40,000, 80,000, 120,000, 160,000, 200,000, and 240,000 cubic feet per second were used. The head at the crest was regulated by means of the drum gates in all but the first six tests, which were made before the gates were installed. With the exception of the last few tests, the flows were regulated as follows: 20,000, 40,000, and 80,000 cubic feet per second with all gates held at elevation 1,034; 120,000 and 160,000 cubic feet per second with all gates set so as to hold

the reservoir at elevation 1,052; 200,000 cubic feet per second with all gates at elevation 1,020; 240,000 cubic feet per second with all gates at elevation 1,020, the flow divided 200,000 cubic feet per second over the spillway and 40,000 cubic feet per second through the conduits. The last tests were made by allowing 40,000 cubic feet per second to pass through the conduits at all flows and regulating the flow over the spillway to 20,000, 40,000, and 80,000 cubic feet per second with all gates held at elevation 1,034; 120,000 and 160,000 cubic feet per second with all gates set at the correct elevation to hold the reservoir at elevation 1,052; 200,000 cubic feet per second with all gates at elevation 1,020, making total quantities of 40,000 (sluices only discharging), 60,000, 80,000, 120,000, 160,000, 200,000, and 240,000 cubic feet per second, respectively. This change was made to follow an operating program developed by the tests.

The action of these tests was recorded pictorially by using still and moving pictures, graphically by use of point gage readings of water surface elevations, and visually by the operator.

For the first six tests, in order to expedite preliminary testing, the powerhouse, drum gates, and outlet conduits were omitted from the model and a sand box extending downstream from the end of the apron was used instead of the regular topography. After the powerhouse and outlet conduits were installed, they were changed from time to time as the designs were altered by the design and testing staffs.

In the original design much thought was given to possible scour on the apron at the toe of the dam. In order to provide for this condition an additional thickness of apron at this point was deemed necessary. The sloping apron, in addition to lessening the excavation in the pool, would provide for this added thickness. Also, the tail water rating curve and the jump-forming height curve for a level depressed apron tend to draw apart at the lower ends (see fig. 332) showing the pool to be too deep at low flows. The sloping apron, by causing the jump to form on the slope, would tend to bring these two curves together, thus giving better conditions at low flows.

In the first model set-up the apron slope was made to conform to the natural dip of the rock strata or an 8:1 slope downstream with a level apron at elevation 810. Although the theoretical analysis showed an apron at elevation 806 necessary to form the jump, the first apron tried was placed at elevation 810 and the top of the sill at the end of the apron at elevation 819. A 60-foot radius bucket connected the downstream face of the dam to the 8:1 sloping portion of the apron. Figure 333 shows this model set-up.

It was found from the first three tests that this arrangement did not give sufficient length of pool for the formation of the jump at low flows, although the action at higher flows was fairly good. The higher discharges show the water surface low over the entire apron and a slight tendency for the pool to "sweep." Considerable scour was noted. When the tail water was raised 4 feet, better results were obtained, thus showing the possibilities of a sloping section of apron with the level portion of the apron at elevation 806.

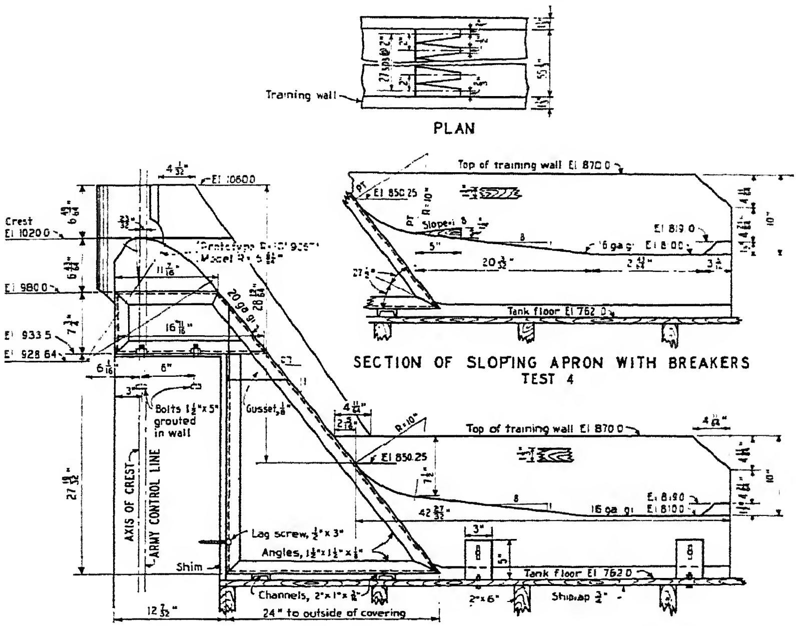
In test No. 4 a set of breaker blocks, shown in figure 333, was added at the point of tangency between the 60-foot radius bucket and the 8:1 slope across the entire width of apron. It was hoped that the jet of water over the dam might be broken up sufficiently to cause more water to remain on the apron and decrease the velocities passing the sill. Results of the test showed, however, that very little additional water was retained on the apron and the velocities over the sill were reduced only slightly. There was slightly less scour downstream from the end of the apron than in the previous tests. The benefits derived from the blocks in reducing the scour and quieting the water surface at high flows were more than offset by the bad conditions at lower flows and probable high initial cost.

The sloping portion of the apron was removed for test No. 5 and the 60-foot radius bucket moved down to connect with a flat apron at elevation 810. This set-up showed insufficient tail water at the intermediate and high flow to insure the formation of a hydraulic jump. Less scour downstream resulted from this set-up than either of the previous tests. By raising the tail water 4 feet the conditions were somewhat improved, although the jump was drowned out to a certain extent at low flows.

In keeping with the indications from previous tests and the action predicted on figure 332, the flat apron was lowered to elevation 806 and connected to the 0.7 slope of the spillway by a 60-foot-radius bucket. The sill at the end of the

apron had a 1:1 slope and a top elevation of 819. The results of this test verified the assumption made from figure 332, and good jump conditions were obtained at all except low flows. The scour conditions with this set-up did not prove serious, and the same set-up was used for tests Nos. 6 and 7.

The bucket was next changed to a 100-foot radius for test No. 8 in order to provide a greater thickness of concrete at the toe of the dam to allow for possible scour at that point. It was hoped that the lower part of the large radius bucket would simulate the sloping apron. With the radius of the bucket increased from 60 to 100 feet, the flow conditions were somewhat improved but the results were not entirely satisfactory. The profile of the water surface showed very little change from conditions with the 60-foot-radius bucket.



SECTION THROUGH CREST SHOWING STEEL BENTS, TRAINING WALL AND SLOPING APRON
TESTS 1, 2 & 3

FIGURE 333.—*Model 8:1 slope, apron at elevation 810, sill at elevation 819.*

The possibilities of using a combination sloping and flat apron were kept in mind, and when the desired results of a larger radius bucket did not materialize, apron slopes of 4:1, 6:1, and 8:1 were tried. The sill at the end of the apron had an upstream slope of 1:1 with a top at elevation 819. A 60-foot-radius bucket was used. The 8:1 and 6:1 slopes did not allow sufficient length of flat apron at elevation 806 to form a good jump, but the 4:1 slope gave good condition throughout the range of flows and much better condition at the low flows than either the flatter slope or the level apron. The 4:1 slope was adopted for further testing.

After fixing the elevation of the flat portion of the apron and the rate of slope for the sloping portion of the apron, a series of tests was conducted to determine the possibility of shortening the length of horizontal apron downstream, or to improve flow conditions by addition or alteration in sills with the same length of apron.

The first change was the slope of the face of the sill at the end of the apron. The original slope was 1:1 from elevation 806 to elevation 819, and the top width was 10 feet. This was changed to a 1½:1 slope with a top width of 5 feet. A comparison of results with the 1½:1 and 1:1 slope shows that less water was held on the apron with the 1½:1 slope sill, and more stable condition of water surface was observed with the 1:1 slope sill.

Next, with this altered sill at the end of the apron an auxiliary sill 6 feet high was tested at one position on the apron, and an auxiliary sill 4 feet high was tested at two positions on the apron. More water was retained on the apron at most flows, but disturbances set up on the apron caused a very undesirable condition in the stilling pool. The sills were also subjected to severe impact by the water flowing at high velocity down the apron, thereby making it highly probable that the sills would suffer considerable scour. The results showed a generally undesirable condition.

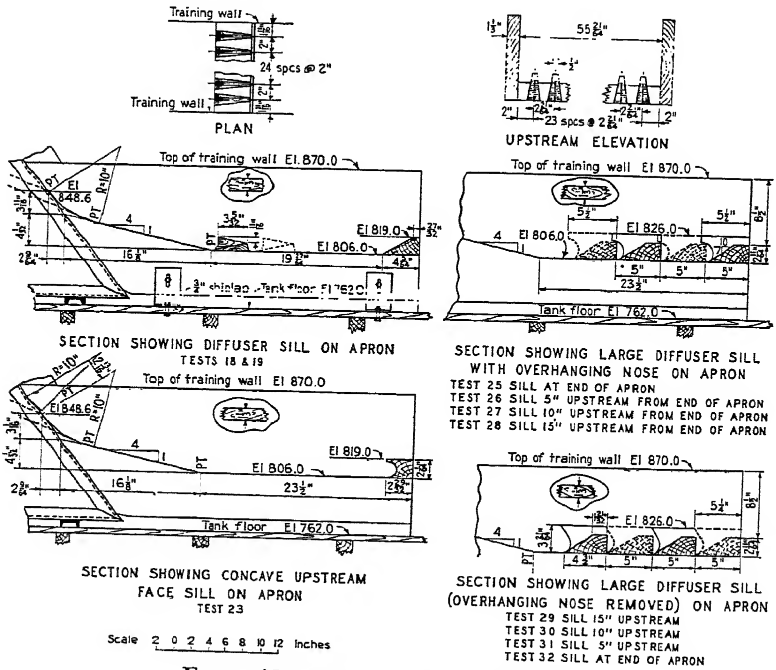


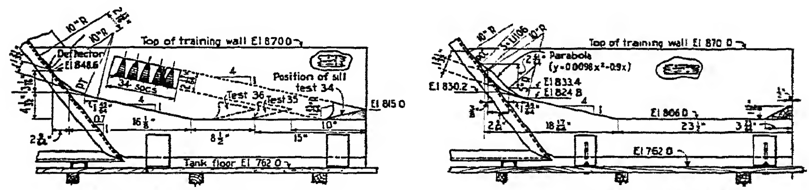
Figure 334.—*Types of diffuser sills tested.*

In an attempt to decrease the height of the sill the top was lowered to elevation 815. A top width of 5 feet and an upstream slope of 1½:1 were still maintained. This lower sill resulted in higher velocities passing the sill and undesirable water surface conditions.

A low diffuser sill was tested in two positions on the apron. (See fig. 334.) The primary purpose of this sill was to spread the jet from the outlet conduits. It was hoped also that flow conditions for the spillway would be improved. This sill was tested at the upstream position with the main sill at the end of the apron, and at the downstream position with the main sill removed. The flow conditions for the spillway were slightly improved, but the blocks were too small to effect a material change in flow conditions, although surface conditions were improved. Very good flow conditions were obtained at low and intermediate discharges.

A sill with a concave upstream face as shown in figure 334 was tested at the downstream end of the apron. It was thought that by using this sill the sheet of high velocity water along the apron would be turned upstream and impinge on the incoming jet. The results showed the jet to be of such depth by the time it reached the sill that it completely filled the concave space and formed in effect a sill with a vertical upstream face. This caused disturbance in the pool directly over the sill and very rough surface conditions elsewhere in the pool. If the radius of curve could have been increased considerably so that the entire high velocity jet would strike the lower half of the curve, the predicted action might have taken place, but this would have necessitated a structure of unreasonably large size.

The concave sill offered possibilities in that the curved upstream face and overhang might be applied to the diffuser-type blocks. Such a sill was tested at different positions on the flat portion of the apron, thus simulating a shortening of the apron. This sill is shown in figure 334. The results were satisfactory in that smoother water surface conditions were obtained and the velocities beyond the end of the apron were very evenly distributed. Paint tests disclosed that part of the jet was turned back on the apron by the curved upstream face of the blocks, thus causing an ellipsoidal roll to form on the apron ahead of the sill. Another portion of the water passed between the blocks and was deflected upward by the sloping portion of the sill and allowed to spread due to the increase in area as the water moved upward and outward. The high velocity jets flowing between the blocks were further broken up and the velocity diminished by the effect of the water spilling off the face of the blocks and into the chamber. Hence, the water after it passed the sill was moving in an upward direction away from the river bed, thus diminishing the possibility of scour downstream from the apron. There was very little scour noted; however, a tendency existed to deposit material at the end of the apron.



A. SECTION SHOWING DIFFUSER SILL (WHEELER TYPE) ON APRON TESTS 34,35 AND 36 – SILL AT END OF APRON 10" AND 15" UPSTREAM RESPECTIVELY

B. SECTION SHOWING PARABOLIC HUMP OVER OUTLET COUNDUITS TEST 42

FIGURE 335.—*A. Wheeler type diffuser sill. B. Parabolic hump over outlet conduits.*

Good results were obtained at all positions of the sill on the apron, although the best results were found when the apron was shortened 60 feet. By use of this sill the apron and training walls could be shortened 60 feet and better flow conditions obtained than with a longer apron. The high initial cost, combined with the probability of damage due to excessive erosion of the sill and impact of logs on the dentates, made the use of this sill inadvisable.

The cost of construction and probability of erosion of the sill were lessened by removing the overhanging noses from the blocks, thus making the faces of the blocks tangent to the curve at 30° to the vertical. This diffuser block is shown in figure 334. This type of block was tested at different positions on the apron. Approximately the same results were obtained as for the preceding tests, except a possible improvement of conditions in the water surface in the pool. Although it was thought that the additional cost of construction of this sill could have been borne by the saving and shortening of the apron and training walls, the bad features of the sill were embodied in the very grave possibility of erosion of the sill and the resulting necessary maintenance.

A low diffuser sill similar to that used at Wheeler Dam was tested at three positions on the apron in an attempt to improve flow conditions. This type sill is shown in figure 335. The results showed an improvement in flow condi-

tions in the pool for low and medium flows and good results with just the sluices operating, but at high spillway flows very little improvement was noted except in surface conditions. The velocities downstream from the end of the apron were much higher than in other set-ups. With the sill at the end of the apron entirely removed, surface conditions agreed closely with the conditions with the diffuser sill at the end of the apron. The sill located 60 feet upstream from the end of the apron produced the best results.

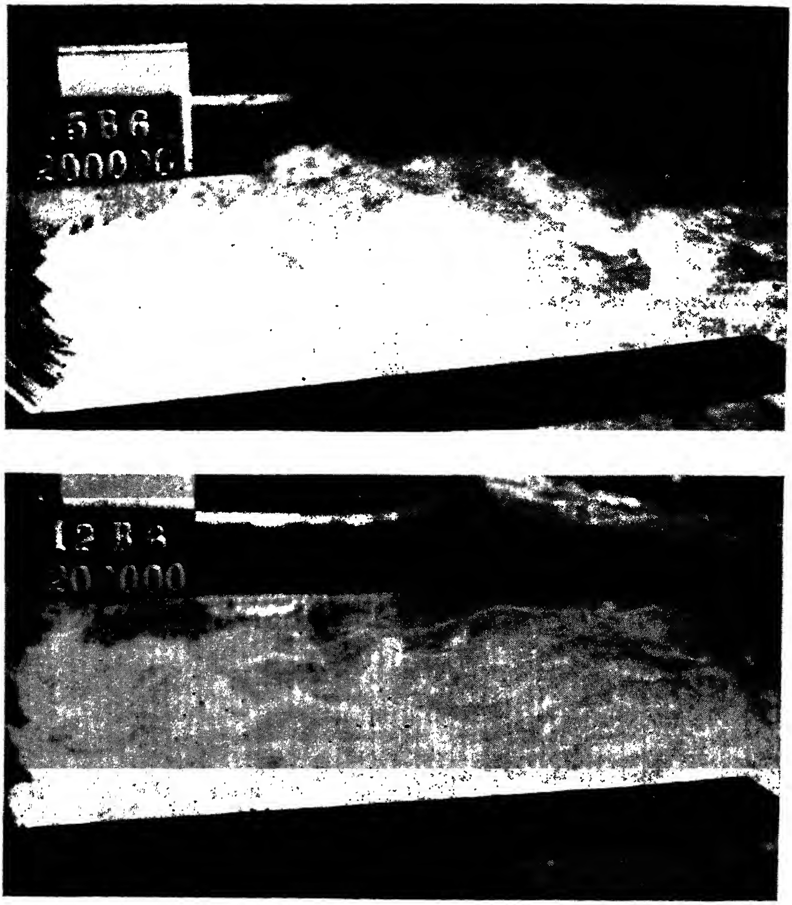


FIGURE 336.—*Comparison of pool action with one of the more unsuccessful model set-ups (A) and the adopted design (B).*

Next, a sill built with a top width of 3 feet and conforming to the contour of the stream bed at the end of the apron for the width of the spillway was tested. The bottom width was held constant at 24 feet and the slope of the upstream face varied accordingly. The velocity of water was concentrated near the center of the apron as it crossed the sill and continued downstream. This was caused

by the lower sill section at the center of the apron and the consequent flatter slope. The higher sill sections at the sides of the spillway apron caused disturbances which resulted in higher water surfaces.

From more accurate cross sections of the stream bed the average elevation of the stream bed was determined to be about elevation 817.5 instead of 819. A set of sills with their tops at that elevation, a constant top width of 3 feet, and upstream face slopes of 1½:1, 2:1, 2½:1, and 3:1 were tested. The results of these tests indicated that the 1½:1 slope sill was the best from the hydraulic standpoint, as well as being less expensive to build. Water surface conditions were improved as compared to the higher sills of the same slope, and no additional erosion was noted at the end of the apron. The modification of the sluice outlet which will be described later made their operation satisfactory with this sill. It was, therefore, recommended for the final design. A parabolic hump as shown in figure 335 on the dam over the outlet conduit opening had very little effect on the flow over the spillway.

Check runs made from time to time showed results of the stilling pool to be very consistent.

Calibration of drum gates.

Prior to the actual calibrating of the drum gates, a tentative schedule of operation was worked out for the gates to cover the range of gate elevations from 1,020 to 1,034.

For each of these gate settings a range of heads over the gates was determined. These heads were not the same for each gate setting but were tentatively determined so that the minimum head was fixed by the elevation of the gates; and the maximum head was that obtained when the elevation of the water surface of the reservoir was approximately 1 foot above the maximum of elevation 1,052.

In the original analysis of the discharge data for the gates the formula $Q=CLH^{3/2}$ was used and the value of the coefficient C was computed in terms of model values for Q, L, and H and with the gates set at elevations 1,020, 1,023.51, 1,027.11, 1,031.54, and 1,033.91, respectively. A summary of these results is shown in table 151. When the values of C were plotted on cross-section paper against the values of H, the resulting curves appeared very erratic so that logarithmic plotting was tried in an effort to straighten the data. Logarithmic plotting produced a good curve for each gate elevation. Difficulty was experienced in this method of analysis due to the short range of values of coefficient C at some gate elevations which made it extremely difficult to read values from the curves.

Since the above method of analysis had the decided disadvantage of not giving smooth curves and therefore making interpolation between observed data difficult, the values of Q and H were then plotted on a logarithmic scale with the equation $Q=KLH^n$ as a basis. The values of K_m and n were obtained from the intercepts and slopes of the straight lines through the data for each gate elevation. Very satisfactory results were produced by this method and to justify it further, additional runs were made with gates set at elevations 1,022.06, 1,025.03, 1,028.93, and 1,031.41, respectively. A decided advantage of this method of analysis was that fewer runs were required to fix the position of a particular curve.

After summarizing the data obtained from the logarithmic plottings, it was found that the values of K_m and n must be fixed with greater accuracy than is possible with logarithmic plotting, due to the slight irregularities of the printing of the paper and the impossibility of interpolating to the necessary degree of accuracy.

Trial computations using the least square method[3] of adjustment of observed data produced a much smaller percentage of error, therefore it was adopted for handling the coefficient data.

Table 152 gives a summary of results of adjusting the data by means of the method of least squares.

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[3] Rohwer, Carl, The Least Square Adjustment of Experimental Data, Bureau of Agricultural Engineering, Department of Agriculture. Appendix II, U. S. Bureau of Reclamation, Technical Memorandum No. 406.

TABLE 151.—*Coefficient of discharge of spillway drum gates*

1	2	3	4	5	6	1	2	3	4	5	6
Test No.	Run No.	Gate elevation	Model Q	Head on model crest	Model C in Q=CLH 3/2	Test No.	Run No.	Gate elevation	Model Q	Head on model crest	Model C in Q=CLH 3/2
6	1	1,020.00	0.431	0.1011	3.224	7	29A	1,027.11	2.732	0.3047	3.899
	2	1,020.00	.918	.1609	3.415		29B	1,027.11	2.747	.3080	3.856
	3	1,020.00	1.839	.2464	3.608		29C	1,027.11	3.614	.3666	3.907
	4	1,020.00	2.725	.3136	3.724		30A	1,027.11	3.629	.3698	3.871
	5	1,020.00	3.625	.3739	3.805		30B	1,027.11	3.616	.3727	3.813
	6	1,020.00	4.556	.4265	3.923		30X	1,027.11	4.532	.4304	3.851
	7	1,020.00	7.901	.6096	3.983		30Y	1,027.11	4.521	.4243	3.924
	8	1,020.00	4.571	.4317	3.866		30Z	1,027.11	4.517	.4280	3.871
	9	1,020.00	4.105	.4035	3.842		37	1,031.54	.228	.0579	3.913
	10	1,020.00	3.147	.3424	3.766		38	1,031.54	.396	.0837	3.931
	11	1,020.00	6.257	.5275	3.917		39	1,031.54	.609	.1119	3.900
	12	1,020.00	7.452	.5914	3.931		40	1,031.54	.842	.1404	3.839
7	7A	1,033.91	.434	.0894	3.898		41	1,031.54	1.114	.1690	3.845
	7X	1,033.91	.226	.0578	3.901		42	1,031.54	1.404	.1950	3.913
	8A	1,033.91	.914	.1476	3.866		43	1,031.54	1.706	.2225	3.898
	9A	1,033.91	1.841	.2381	3.803		44	1,031.54	2.080	.2536	3.908
	9X	1,033.91	1.343	.1916	3.842		45	1,031.54	2.270	.2690	3.904
	9Y	1,033.91	1.349	.1918	3.853		46	1,020.00	.459	.1026	3.346
	10A	1,033.91	2.725	.3083	3.820		47	1,020.00	.917	.1586	3.482
	10B	1,033.91	3.677	.3761	3.825		48	1,020.00	1.846	.2455	3.642
	10C	1,033.91	3.162	.3424	3.787		49	1,020.00	2.746	.3144	3.737
	10D	1,033.91	3.154	.3392	3.830		50	1,020.00	3.655	.3695	3.904
	10E	1,033.91	3.671	.3748	3.839		51	1,020.00	4.531	.4303	3.851
	10X	1,033.91	2.286	.2749	3.805		52	1,020.00	5.349	.4785	3.878
	21	1,031.41	.218	.0573	3.819		53	1,020.00	6.218	.5243	3.931
	22	1,031.41	.433	.0892	3.910		50A	1,020.00	3.716	.3773	3.847
	23	1,031.41	.911	.1465	3.894		51A	1,020.00	4.522	.4340	3.796
	24	1,031.41	1.837	.2349	3.872		52A	1,020.00	5.381	.4753	3.941
	25	1,031.41	2.734	.3060	3.874		47A	1,020.00	.928	.1610	3.448
	21X	1,031.56	.323	.0725	3.980		46X	1,020.00	.229	.0661	3.222
	22A	1,031.56	.430	.0873	3.994		50B	1,020.00	3.617	.3731	3.808
	22B	1,031.56	.435	.0880	4.002		54	1,028.93	.438	.0902	3.876
	22C	1,031.56	.435	.0889	3.942		55	1,028.93	.929	.1493	3.864
	22X	1,031.56	.941	.1501	3.884		56	1,028.93	1.852	.2359	3.880
	22Y	1,031.56	.681	.1203	3.911		57	1,028.93	2.723	.3039	3.901
	23A	1,031.56	.913	.1468	3.894		58	1,025.03	.924	.1544	3.658
	23X	1,031.56	1.380	.1930	3.906		59	1,025.03	1.851	.2427	3.715
	24A	1,031.56	1.843	.2339	3.909		60	1,025.03	2.753	.3125	3.782
	24X	1,031.56	2.229	.2648	3.926		61	1,025.03	3.632	.3752	3.793
	25A	1,031.56	2.716	.3025	3.915		62	1,025.03	.438	.0952	3.582
	25B	1,031.56	2.716	.3021	3.923		63	1,022.06	.436	.1006	3.283
	26	1,026.91	.441	.0953	3.596		64	1,022.06	.923	.1615	3.413
	27	1,026.91	.916	.1519	3.709		65	1,022.06	1.470	.2167	3.496
	28	1,026.91	1.828	.2377	3.785		66	1,022.06	3.600	.3795	3.696
	29	1,026.91	2.724	.3087	3.810		67	1,022.06	4.526	.4330	3.813
	30	1,026.91	3.635	.3725	3.836		68	1,023.51	.441	.0997	3.362
	26A	1,027.11	.434	.0919	3.737		69	1,023.51	.922	.1594	3.477
	27A	1,027.11	.925	.1505	3.800		70	1,023.51	1.837	.2459	3.615
	28A	1,027.11	1.838	.2360	3.849		71	1,023.51	3.670	.3822	3.713

Table 152.—*Adjustment by least square method*

1	2	3	4	5	6	7	8	9
Test No.	Run No.	Gate elevation	Model head on crest	Model Q	Colog H	Log Q	(Log H) ²	Log H × log Q
6	1	1020.00	0.1011	0.431	−0.9953	−0.3652	0.9905	+0.3635
	2	1020.00	.1609	.918	−.7934	−.0372	.6295	+.0295
	3	1020.00	.2464	1.839	−.6084	+.2645	.3701	−.1609
	4	1020.00	.3136	2.723	−.5036	+.4350	.2536	−.2191
	5	1020.00	.3739	3.625	−.4272	+.5593	.1825	−.2389
	7	1020.00	.6096	7.901	−.2150	+.8977	.0462	−.1930
	8	1020.00	.4317	4.571	−.3648	+.6600	.1331	−.2408
	9	1020.00	.4035	4.105	−.3942	+.6133	.1554	−.2417
	10	1020.00	.3424	3.147	−.4655	+.4979	.2167	−.2317
	11	1020.00	.5275	6.257	−.2778	+.7964	.0772	−.2212
	12	1020.00	.5914	7.452	−.2281	+.8723	.0520	−.1990
					−5.2733	+5.1940	3.1068	−1.5533

$11\,A = 5.2733\,n + 5.1940$ $\quad 1.5533 + 3.1068\,n = 5.2733\,A$

$A = 1.2476 = \text{Log}\,K_m\,L$ $\quad n = 1.6175$

$K_m\,L = 17.68$ $\quad L = 4.1668$ $\quad K_m = 4.244$

1	2	3	4	5	6	7	8	9
7	46	1020.00	0.1026	0.459	−0.9889	−0.3383	0.9778	+0.3345
	46X	1020.00	.0661	.229	−1.1798	−.6407	1.3919	+.7559
	47	1020.00	.1586	.917	−.7997	−.0379	.6395	+.0303
	47A	1020.00	.1610	.928	−.7932	−.0323	.6291	+.0256
	48	1020.00	.2455	1.846	−.6100	+.2663	.3720	−.1624
	49	1020.00	.3144	2.746	−.5025	+.4387	.2525	−.2204
	50	1020.00	.3695	3.655	−.4324	+.5629	.1870	−.2434
	50A	1020.00	.3773	3.716	−.4233	+.5700	.1792	−.2413
	50B	1020.00	.3731	3.617	−.4282	+.5583	.1833	−.2391
	51	1020.00	.4303	4.531	−.3662	+.6562	.1341	−.2403
	51A	1020.00	.4340	4.522	−.3625	+.6553	.1314	−.2376
	52	1020.00	.4785	5.349	−.3201	+.7283	.1025	−.2331
	52A	1020.00	.4753	5.381	−.3230	+.7309	.1044	−.2361
	53	1020.00	.5243	6.218	−.2804	+.7937	.0786	−.2226
					−7.8102	+4.9114	5.3633	−1.1300

$14\,A = 7.8102\,n + 4.9114$ $\quad 5.3633\,n + 1.1300 = 7.8102\,A$

$A = 1.2432 = \text{Log}\,K_m\,L$ $\quad n = 1.5997$

$K_m\,L = 17.51$ $\quad L = 4.1668$ $\quad K_m = 4.202$

1	2	3	4	5	6	7	8	9
7	63	1022.06	0.1006	0.436	−0.9974	−0.3602	0.9948	+0.3592
	64	1022.06	.1615	.923	−.7918	−.0349	.6270	+.0276
	65	1022.06	.2167	1.470	−.6642	+.1673	.4411	−.1111
	66	1022.06	.3795	3.600	−.4208	+.5563	.1771	−.2341
	67	1022.06	.4330	4.526	−.3635	+.6558	.1321	−.2384
					−3.2377	+.9843	2.3721	−.1968

$5\,A = 3.2377\,n + 0.9843$ $\quad 0.1968 + 2.3721\,n = 3.2377\,A$

$A = 1.2323 = \text{Log}\,K_m\,L$ $\quad n = 1.5990$

$K_m\,L = 17.07$ $\quad L = 4.1670$ $\quad K_m = 4.097$

1	2	3	4	5	6	7	8	9
7	68	1023.51	0.0997	0.441	−1.0013	−0.3556	1.0026	+0.3560
	69	1023.51	.1594	.922	−.7975	−.0353	.6360	+.0282
	70	1023.51	.2459	1.837	−.6092	+.2641	.3712	−.1609
	71	1023.51	.3832	3.670	−.4166	+.5647	.1735	−.2352
					−2.8246	+.4379	2.1833	−.0119

$4\,A = 2.8246\,n + 0.4379$ $\quad 2.1833\,n + 0.0119 = 2.8246\,A$

$A = 1.2220 = \text{Log}\,K_m\,L$ $\quad n = 1.5754$

$K_m\,L = 16.67$ $\quad L = 4.1670$ $\quad K_m = 4.001$

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TABLE 152.—*Adjustment by least square method*—Continued

1	2	3	4	5	6	7	8	9
Test No.	Run No.	Gate elevation	Model head on crest	Model Q	Colog H	Log Q	(Log H)[2]	Log H × log Q
7	58	1025.03	0.1544	0.924	−0.8114	−0.0343	0.6583	+0.0278
	59	1025.03	.2427	1.851	−.6149	+.2674	.3781	−.1644
	60	1025.03	.3125	2.753	−.5052	+.4398	.2552	−.2222
	61	1025.03	.3752	3.632	−.4257	+.5601	.1813	−.2385
	62	1025.03	.0952	.438	−1.0214	−.3581	1.0432	+.3658
					−3.3786	+.8749	2.5161	−.2315

$5A = 3.3786\,n + 0.8749$ $\qquad 2.5161\,n + 0.2315 = 3.3786\,A$

$A = 1.2175 = \text{Log}\,K_m L$ $\qquad n = 1.5429$

$K_m L = 16.50 \qquad L = 4.1670$ $\qquad K_m = 3.960$

1	2	3	4	5	6	7	8	9
7	26A	1027.11	0.0919	0.434	−1.0367	−0.3625	1.0747	+0.3758
	27A	1027.11	.1505	.925	−.8225	−.0338	.6764	+.0278
	28A	1027.11	.2360	1.838	−.6271	+.2645	.3932	−.1658
	29A	1027.11	.3047	2.732	−.5161	+.4366	.2664	−.2253
	29B	1027.11	.3080	2.747	−.5115	+.4388	.2616	−.2244
	29C	1027.11	.3666	3.614	−.4358	+.5580	.1899	−.2432
	30A	1027.11	.3698	3.629	−.4320	+.5598	.1867	−.2419
	30B	1027.11	.3727	3.616	−.4286	+.5582	.1837	−.2393
	30X	1027.11	.4304	4.532	−.3661	+.6563	.1341	−.2403
	30Y	1027.11	.4243	4.521	−.3723	+.6552	.1386	−.2440
	30Z	1027.11	.4280	4.517	−.3686	+.6549	.1358	−.2414
					−5.9173	+4.3860	3.6411	−1.6620

$11A = 5.9173\,n + 4.3859$ $\qquad 3.6411\,n + 1.6620 = 5.9173\,A$

$A = 1.2178 = \text{Log}\,K_m L$ $\qquad n = 1.5227$

$K_m L = 16.51 \qquad L = 4.1670$ $\qquad K_m = 3.963$

1	2	3	4	5	6	7	8	9
7	54	1028.93	0.0902	0.438	+1.0448	+0.3585	1.0916	+0.3746
	55	1028.93	.1493	.929	.8259	+.0319	.6822	+.0264
	56	1028.93	.2359	1.852	.6273	−.2677	.3935	−.1679
	57	1028.93	.3039	2.723	.5173	−.4351	.2676	−.2251
					3.0153	−.3124	2.4349	+.0080

$4A = 3.0153\,n + 0.3124$ $\qquad 2.4349\,n - 0.0080 = 3.0153\,A$

$A = 1.2119 = \text{Log}\,K_m L$ $\qquad n = 1.5041$

$K_m L = 16.29 \qquad L = 4.1670$ $\qquad K_m = 3.909$

1	2	3	4	5	6	7	8	9
7	21	1031.41	0.0573	0.218	−1.2418	−0.6609	1.5422	+0.8208
	22	1031.41	.0892	.433	−1.0496	−.3631	1.1017	+.3811
	23	1031.41	.1465	.911	−.8342	−.0407	.6958	+.0340
	24	1031.41	.2349	1.837	−.6291	+.2641	.3958	−.1661
	25	1031.41	.3060	2.734	−.5143	+.4368	.2645	−.2246
					−4.2690	−.3638	4.0000	+.8452

$5A = 4.2690\,n - 0.3638$ $\qquad 4.0000\,n - 0.8452 = 4.2690\,A$

$A = 1.2123 = \text{Log}\,K_m L$ $\qquad n = 1.5051$

$K_m L = 16.30 \qquad L = 4.1670$ $\qquad K_m = 3.913$

1	2	3	4	5	6	7	8	9
7	37	1031.54	0.0579	0.228	−1.2373	−0.6426	1.5310	+0.7952
	38	1031.54	.0837	.396	−1.0773	−.4021	1.1605	+.4332
	39	1031.54	.1119	.609	−.9512	−.2156	.9047	+.2050
	40	1031.54	.1404	.842	−.8526	−.0749	.7270	+.0638
	41	1031.54	.1690	1.114	−.7721	+.0468	.5962	−.0361
	42	1031.54	.1950	1.404	−.7100	+.1475	.5041	−.1047
	43	1031.54	.2225	1.706	−.6527	+.2321	.4260	−.1515
	44	1031.54	.2536	2.080	−.5959	+.3181	.3550	−.1896
	45	1031.54	.2690	2.270	−.5703	+.3560	.3252	−.2030
					−7.4194	−.2347	6.5297	+.8123

$9A = 7.4194\,n - 0.2347$ $\qquad 6.5297\,n - 0.8123 = 7.4194\,A$

$A = 1.2078 = \text{Log}\,K_m L$ $\qquad n = 1.4967$

$K_m L = 16.13 \qquad L = 4.1670$ $\qquad K_m = 3.872$

TABLE 152.—*Adjustment by least square method*—Continued

1	2	3	4	5	6	7	8	9
Test No.	Run No.	Gate elevation	Model head on crest	Model Q	Colog H	Log Q	(Log H)	Log H × log Q
7	21X	1031.56	0.0725	0.323	−1.1397	−0.4902	1.2988	+0.5587
	22A	1031.56	.0873	.430	−1.0590	−.3668	1.1215	+.3884
	22B	1031.56	.0880	.435	−1.0555	−.3611	1.1141	+.3812
	22C	1031.56	.0889	.435	−1.0511	−.3618	1.1048	+.3803
	22X	1031.56	.1501	.941	−.8236	−.0262	.6784	+.0216
	22Y	1031.56	.1203	.681	−.9197	−.1672	.8459	+.1537
	23A	1031.56	.1468	.913	−.8333	−.0394	.6943	+.0328
	23X	1031.56	.1930	1.380	−.7144	+.1400	.5104	−.1000
	24A	1031.56	.2339	1.843	−.6310	+.2656	.3981	−.1676
	24X	1031.56	.2648	2.229	−.5771	+.3481	.3330	−.2009
	25A	1031.56	.3025	2.716	−.5193	+.4339	.2696	−.2253
	25B	1031.56	.3021	2.716	−.5199	+.4339	.2702	−.2255
					−9.8436	−.1912	8.6391	+.9974
		$12A = 9.8436n - 0.1912$ $A = 1.2049 = \text{Log } K_m L$ $K_m L = 16.03 \quad L = 4.1670$			$8.6391n - 0.9974 = 9.8436A$ $n = 1.4883$ $K_m = 3.847$			
7	7A	1033.91	0.0894	0.434	−1.0487	−0.3627	1.0997	+1.4290
	7X	1033.91	.0578	.226	−1.2381	−.6453	1.5328	+.7989
	8A	1033.91	.1476	.914	−.8309	−.0392	.6904	+.0326
	9A	1033.91	.2381	1.841	−.6232	+.2652	.3884	−.1653
	9X	1033.91	.1916	1.343	−.7176	+.1281	.5150	−.0919
	9Y	1033.91	.1918	1.349	−.7172	+.1299	.5143	−.0931
	10A	1033.91	.3083	2.725	−.5110	+.4354	.2612	−.2225
	10B	1033.91	.3761	3.677	−.4247	+.5655	.1804	−.2402
	10C	1033.91	.3424	3.162	−.4655	+.5000	.2167	−.2327
	10D	1033.91	.3392	3.154	−.4695	+.4989	.2205	−.2343
	10E	1033.91	.3748	3.671	−.4262	+.5648	.1817	−.2407
	10Y	1033.91	.2749	2.286	−.5608	+.3590	.3145	−.2013
					−8.0334	+2.3996	6.1156	−.5101
		$12A = 8.0334n + 2.3996$ $A = 1.1951 = \text{Log } K_m L$ $K_m L = 15.67 \quad L = 4.1670$			$6.1156n + 0.5101 = 8.0334A$ $n = 1.4864$ $K_m = 3.760$			

The runs were divided into groups, made with the gates at the same elevation. At the bottom of each of these groups the calculations for the values of K_m and for n for that elevation are shown.

An explanation of the laws of hydraulic similitude and the transference of results from model to prototype are given below.[4] The subscript $_p$ denotes prototype quantities, and the subscript $_m$ denotes model quantities. The scale ratio of the model is denoted by N. On page 695 it was stated that

$$Q_p = Q_m N^{5/2} \tag{1}$$

also, in the case of the quantity of water flowing over a dam

$$Q_p = K_p L_p H_p^{n_p} \tag{2}$$

and, for the model

$$Q_m = K_m L_m H_m^{n_m} \tag{3}$$

from (1) and (3)

$$Q_p = K_m L_m H_m^{n_m} N^{5/2} \tag{4}$$

equating (2) and (4)

$$K_p L_p H_p^{n_p} = K_m L_m H_m^{n_m} N^{5/2} \tag{5}$$

but

$$L_p = N L_m \quad H_p = N H_m$$

[4] Chick, Alton C., John R. Freeman's Hydraulic Laboratory Practice, p. 796.

substituting in (5), assuming $n_p = n_m = n$

$$K_p N L_m (N H_m)^n = K_m L_m H_m{}^n N^{1/2}$$

simplifying

$$K_p = K_m N^{3/2-n} \qquad (6)$$

With the value of K_m and n for each gate setting a table of values for K_p and n were obtained and are shown in table 153. Values of K_p were calculated using equation (6) and the values of K_m and n from table 152. The values of n are the same whether model or prototype. Table 153 gives the calculated results. These data were plotted as shown on figure 337.

TABLE 153.—*Calculations of prototype coefficient of discharge from observed data*

$K_{proto} = K_{model} \times N^{1.5-n}$ where $N=72$ and Log $N=1.8573$

1	2	3	4	5	6	7
Gate elevation	See table 152 n	1.5–n	Log N× (1.5–n)	$N^{1.5-n}$	K Model	K Prototype
1020.00	1.618	−0.118	−0.2182	0.6050	4.244	2.568
1020.00	1.600	−.100	−.1852	.6529	4.202	2.743
1022.06	1.600	−.100	−.1839	.6548	4.097	2.683
1023.51	1.575	−.075	−.1400	.7244	4.001	2.898
1025.03	1.543	−.043	−.0797	.8324	3.960	3.296
1027.11	1.523	−.023	−.0422	.9075	3.963	3.596
1028.93	1.504	−.004	−.0076	.9826	3.909	3.841
1031.41	1.505	−.005	−.0095	.9784	3.913	3.828
1031.54	1.497	+.003	+.0061	1.0142	3.872	3.927
1031.56	1.488	+.012	+.0217	1.0513	3.847	4.044
1033.91	1.487	+.013	+.0251	1.0594	3.760	3.984

All the curves in figure 337 are interdependent. For instance, if the prototype coefficient curve is changed slightly, then the other three curves must be changed accordingly in order for the given values to check. All the plotted points are not given the same weight on the curves as there are more observations on some than on others.

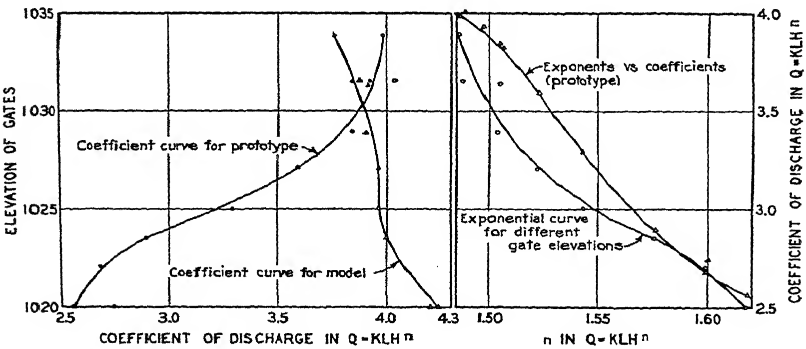


FIGURE 337.—*Coefficient* (K) *and exponential* (n) *curves for* $Q=KLH^n$

Values of K_p and n were read from these curves for each foot of gate elevation. By substituting these values in the equation $Q=K_pLH^n$, and using different values of H, the discharge table, table 154, was prepared.

Table 154 was compiled to furnish material for the construction of discharge curves. Therefore, only a sufficient number of points was calculated to fix the curves. Additional points may be calculated from the values given at the

head of the column. The results in this table were plotted and are shown in figure 338. This diagram is based on having all the gate crests at the same elevation.

TABLE 154.—*Discharge table computed from the formula $Q=KLH^n$*

[Discharge (Q) for all three gates at the same elevation]

Head over gates in feet	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation
	Gate elevation 1,020 $n=1.617$ $K_p=2.568$ $K_pL=770.4$		Gate elevation 1,021 $n=1.608$ $K_p=2.617$ $K_pL=785.1$		Gate elevation 1,022 $n=1.597$ $K_p=2.696$ $K_pL=808.8$		Gate elevation 1,023 $n=1,584$ $K_p=2.809$ $K_pL=842.7$		Gate elevation 1,024 $n=1.566$ $K_p=3.002$ $K_pL=900.6$	
1	770	1,021	785	1,022	809	1,023	843	1,024	901	1,025
2	2,364	1,022	2,393	1,023	2,447	1,024	2,527	1,025	2,667	1,026
3	4,544	1,023	4,594	1,024	4,676	1,025	4,804	1,026	5,032	1,027
4	7,252	1,024	7,296	1,025	7,404	1,026	7,577	1,027	7,895	1,028
6	13,970	1,026	14,010	1,027	14,150	1,028	14,400	1,029	14,900	1,030
8	22,250	1,028	22,240	1,029	22,400	1,030	22,720	1,031	23,380	1,032
10	31,920	1,030	31,840	1,031	31,990	1,032	32,360	1,033	33,150	1,034
12	42,860	1,032	42,690	1,033	42,810	1,034	43,190	1,035	44,110	1,036
14	55,000	1,034	54,700	1,035	54,760	1,036	55,140	1,037	56,150	1,038
16	68,260	1,036	67,810	1,037	67,770	1,038	68,130	1,039	69,210	1,040
20	97,920	1,040	97,080	1,041	96,790	1,042	97,030	1,043	98,160	1,044
24	131,500	1,044	130,100	1,045	129,500	1,046	129,500	1,047	130,600	1,048
28	168,700	1,048	166,800	1,049	165,700	1,050	165,400	1,051	166,300	1,052
32	209,400	1,052	206,700	1,053	205,100	1,054	204,300	1,055	---	---
36	253,300	1,056	---	---	---	---	---	---	---	---

Head over gates in feet	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation
	Gate elevation 1,025 $n=1.549$ $K_p=3.220$ $K_pL=966.0$		Gate elevation 1,026 $n=1.535$ $K_p=3.410$ $K_pL=1,023$		Gate elevation 1,027 $n=1.524$ $K_p=3.574$ $K_pL=1,072$		Gate elevation 1,028 $n=1.515$ $K_p=3.704$ $K_pL=1,111$		Gate elevation 1,029 $n=1.508$ $K_p=3.796$ $K_pL=1,139$	
1	966	1,026	1,023	1,027	1,072	1,028	1,111	1,029	1,139	1,030
2	2,826	1,027	2,965	1,028	3,084	1,029	3,176	1,030	3,239	1,031
3	5,294	1,028	5,524	1,029	5,720	1,030	5,870	1,031	5,970	1,032
4	8,266	1,029	8,591	1,030	8,868	1,031	9,076	1,032	9,212	1,033
6	15,490	1,031	16,010	1,032	16,450	1,033	16,780	1,034	16,980	1,035
8	24,180	1,033	24,900	1,034	25,500	1,035	25,940	1,036	26,200	1,037
10	34,160	1,035	35,070	1,036	35,830	1,037	36,370	1,038	36,680	1,039
12	45,300	1,037	46,390	1,038	47,310	1,039	47,950	1,040	48,290	1,041
14	57,510	1,039	58,770	1,040	59,840	1,041	60,560	1,042	60,930	1,043
16	70,720	1,041	72,140	1,042	73,340	1,043	74,140	1,044	74,520	1,045
20	99,910	1,045	101,600	1,046	103,100	1,047	104,000	1,048	104,300	1,049
24	132,500	1,049	134,400	1,050	136,100	1,051	137,000	1,052	137,300	1,053
28	168,200	1,053	170,300	1,054	172,100	1,055	---	---	---	---

Head over gates in feet	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation	Quantity in cubic feet per second	Reservoir water surface elevation
	Gate elevation 1,030 $n=1.502$ $K_p=3.865$ $K_pL=1,160$		Gate elevation 1,031 $n=1.497$ $K_p=3.918$ $K_pL=1,175$		Gate elevation 1,032 $n=1.493$ $K_p=3.951$ $K_pL=1,185$		Gate elevation 1,033 $n=1.489$ $K_p=3.972$ $K_pL=1,192$		Gate elevation 1,034 $n=1.486$ $K_p=3.985$ $K_pL=1.196$	
1	1,160	1,031	1,175	1,032	1,185	1,033	1,192	1,034	1,196	1,035
2	3,284	1,032	3,318	1,033	3,336	1,034	3,345	1,035	3,349	1,036
3	6,038	1,033	6,089	1,034	6,112	1,035	6,119	1,036	6,117	1,037
4	9,302	1,034	9,367	1,035	9,391	1,036	9,392	1,037	9,380	1,038
6	17,100	1,036	17,190	1,037	17,200	1,038	17,180	1,039	17,140	1,040
8	26,350	1,038	26,440	1,039	26,430	1,040	26,370	1,041	26,280	1,042
10	36,840	1,040	36,930	1,041	36,880	1,042	36,770	1,043	36,610	1,044
12	48,440	1,042	48,520	1,043	48,420	1,044	48,230	1,045	48,000	1,046
14	61,060	1,044	61,120	1,045	60,950	1,046	60,680	1,047	60,350	1,048
16	74,620	1,046	74,640	1,047	74,400	1,048	74,030	1,049	73,600	1,050
20	104,300	1,050	104,300	1,051	103,800	1,052	103,200	1,053	102,500	1,054
24	137,200	1,054	137,000	1,055	---	---	---	---	---	---

The results of the coefficient test show that the spillway will discharge the quantity anticipated in the preliminary design calculations and have a small margin of safety in the form of additional discharge.

A discharge curve for the prototype spillway was the major requirement of the coefficient experiment, but in order to show the results in a more conventional form the values of C in the formula $Q=CLH^{3/2}$ as shown in table 151 were plotted against reservoir elevations. In order to obtain a smooth curve through these observed points on cross-section paper, especially when there were

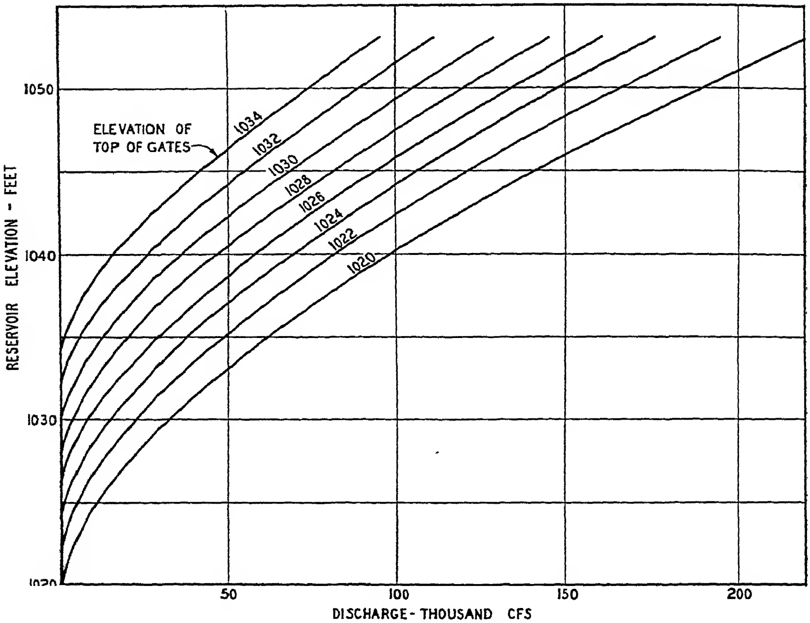


FIGURE 338.—*Discharge curves.*

only four or five observations, values of C for each curve were calculated. For each observed gate elevation, values of the head covering the observed range were assumed and values of C calculated by use of the formula $C=K_pH_p^{n-3/2}$. This equation is derived as follows:

$$Q_p=CL_pH_p^{3/2} \text{ and } Q_p=K_pL_pH_p^{n} \qquad (1) \text{ and } (2)$$

equating (1) and (2)

$$CL_pH_p^{3/2}=K_pL_pH_p^{n} \qquad (3)$$

or

$$C=K_pH_p^{n-3/2} \qquad (4)$$

The values of K_p and n were taken from the curves on figure 337. Curves were then drawn through these calculated points. As can be seen on figure 339, these are smooth curves and pass through the observed points very consistently, but there is no way of interpolating results for gate elevations between those observed directly.

By the above method, curves showing C plotted against reservoir elevation for gate elevations of 1,020, 1,022, and so forth, were plotted and are shown on

figure 340. These curves were constructed to make possible a comparison between the results of these tests and tests on other models that have been calculated for C in $Q=CLH^{3/2}$.

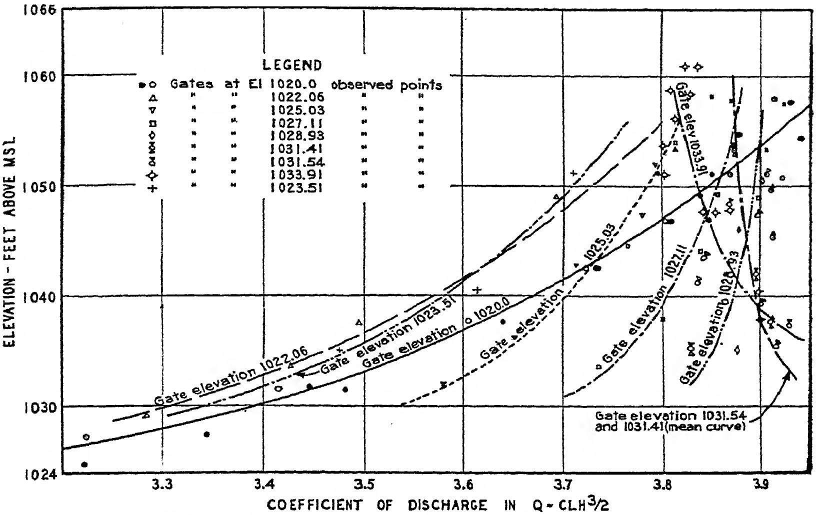


FIGURE 339.—*Coefficient of discharge (C) in $Q=CLH^{3/2}$—Reservoir elevation for com parison with observed data.*

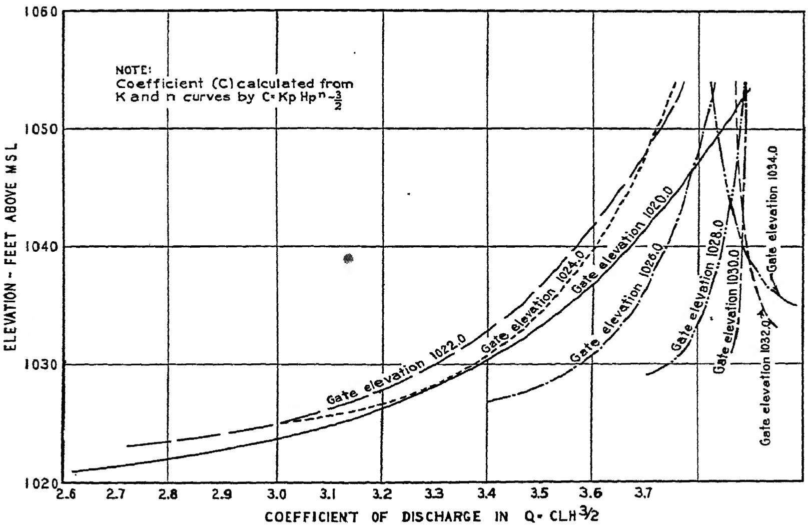


FIGURE 340.—*Coefficient of discharge (C) in $Q=CLH^{3/2}$—Reservoir elevation for comparison with other similar data.*

The fact that the curves cross each other can be explained by a study of the changing crest section as the gates are raised. The angle between the slope on the upstream face of the crest section and the chord of the gate section increases very rapidly as the gates are raised, thus changing the conditions of flow radically.

All results were checked, and the plotted curves show the final analysis of the data.

The values of C in $Q=CLH^{3/2}$ are assumed to be the same for model and prototype.

Pressures on drum gates.

Eight piezometer openings were installed in one of the drum gates and connected to glass tubes by means of rubber tubing. These piezometer openings were referenced to the gage board in such a manner that the capillary attraction in the glass tube was taken into account. The pressures were read in feet of water directly on the glass gage tubes. While the above-mentioned calibration runs were in progress, a series of readings on the gage tubes were observed for each gate elevation and head over the gate. These results were transferred to prototype values and plotted. The curves shown in figure 341 are representative of the data obtained.

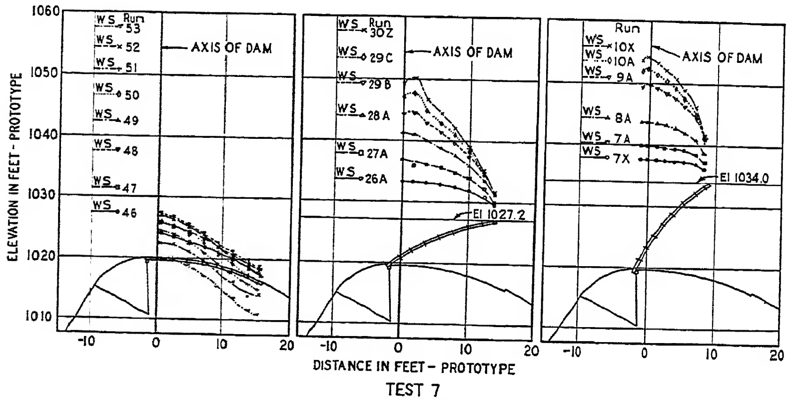


Figure 341.—*Drum gate pressures.*

As no pressures were found to be substantially less than atmospheric, the results could be transferred by simply multiplying model pressures by the scale ratio (72).

Tests on training walls.

From visual observations of the upper nappe of the jet along the training wall on the downstream face of the dam, it was felt that these walls might be modified to contain less yardage of concrete and improve their architectural appearance. Upper nappe measurements were made with all gates at elevation 1,034 and the reservoir surface at elevation 1,052 and all gates set so as to hold the reservoir at elevation 1,052 when quantities equal to 120,000, 160,000, and 180,000 cubic feet per second were passing the spillway.

The results of the test show that the upper corner of the training wall could be reduced to a curve of approximately 35-foot radius and result in a saving of concrete. The point of intersection of the sloping training wall with the horizontal wall could be moved closer to the downstream face of the dam, thus making a steeper slope than that given and would result in a further saving.

It was also desired to obtain pressures on the sides of the training wall on the power house side of the apron to assist in the design of this wall. Twenty-five piezometer openings were placed on the spillway side of the training wall

and connected to glass gage tubes by means of rubber tubing. Runs of 20,000, 40,000, 60,000, 80,000, 120,000, 160,000, 200,000, 240,000 cubic feet per second were made, and a series of observations made on the gage tubes for each run. A representative group of results together with the location of the openings are shown in figure 342.

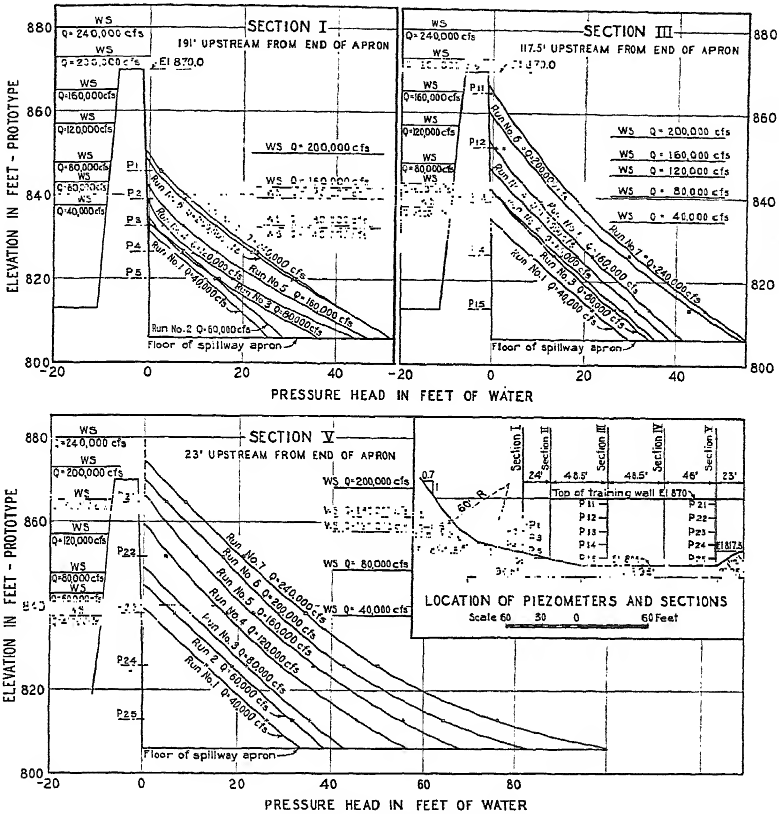


Figure 342.—*Training wall pressures.*

At higher discharges the section of wall from the front face of the powerhouse extending downstream is in effect an overflow dam. For all discharges above 185,000 cubic feet per second, water flows over the top of the wall from the powerhouse tailrace into the stilling pool and at 240,000 cubic feet per second reaches a depth of 10 feet. This action is shown to be maximum at section 4, and there is a slight negative pressure near the top of the wall. The water surface in the stilling pool is high enough at section 5 to make the effect negligible. There is, however, a flow over the wall into the stilling pool from the front edge of the powerhouse to the end of the apron.

Velocity studies.

Measurements of the velocities on the apron of the model were made for comparison with the computed velocities on the prototype to determine the magnitude of the error which results from the fact that the model might not

have relatively the same coefficient of roughness as the prototype. The results of the comparison of the velocities measured on the model with those computed from the prototype showed only a small difference between observed and computed values. Table 155 gives a summation of the results of the velocity studies.

TABLE 155.—*Comparison of velocities measured on model and converted to prototype and velocities computed from prototype*

Quantity in cubic feet per second	Theoretical		Friction loss in velocity head feet of water		Net				Percent difference calculated and measured velocities
					Velocity F. p. s.		Velocity head Ft.		
	Velocity F. p. s.	Velocity head Ft.	Computed	Measured	Computed	Measured	Computed	Measured	
1	2	3	4	5	6	7	8	9	10
40,000	122.6	233.9	92.9	102.2	95.2	91.6	141.0	130.6	3.74
120,000	124.8	242.0	38.5	41.4	114.4	112.6	203.5	197.1	1.61
200,000	124.4	241.0	22.5	20.6	118.4	117.7	218.5	215.5	0.56

Gate operating program.

For the purpose of determining a gate operating program, a series of runs was made with the gates set at different elevations from the fixed crests. All combinations of one gate at elevation 1,020 and the other two at elevation 1,034 were run with the reservoir elevation held at elevation 1,034 and also at 1,052. Similarly, all combinations of two gates at elevation 1,020 and one at elevation 1,034 were run with the reservoir held at elevation 1,034 and also at elevation 1,052.

Results show that when the gates are at widely different elevations the conditions in the stilling pool are not satisfactory. The jets from the gates at widely different elevations combine on the rollway and set up a disturbance before the water enters the pool. For instance, with the center gate up and the two outside gates down, the jets from the outside gates spread in under the jet from the center gate because it strikes farther down on the rollway. This disturbance is carried down through the pool.

In testing for an operating program the possibility of some irregularity that might cause one or more of the gates to become inactive was considered. For instance, a test was made of the action with gates 1 and 2 at elevation 1,034 and gate 3 at elevation 1,020. The results show bad conditions in the pool but not serious enough to cause alarm over the safety of the structure.

The most undesirable condition resulted when the tail water was at a minimum, the reservoir at elevation 1,034, all gates at elevation 1,034, and one of them suddenly dropped to elevation 1,020. This allowed approximately 18,000 cubic feet per second of water to rush into the pool. Although the tail water was swept off the sloping apron and into the flat part of the apron, whirls soon developed which prevented the tail water from being swept past the sill. The tail water very quickly filled up sufficiently to control the jump.

Undesirable conditions were caused when the gate farthest from the powerhouse was completely down and the center gate and gate nearest the powerhouse were completely raised. This condition results because there is more area in the whirl when it turns toward the powerhouse rather than away from it. A larger whirl is of more severe proportions. With the gate nearest the powerhouse completely down and the other two completely raised, the whirl beyond the end of the apron can turn both into the tailrace and toward the right bank and is somewhat dissipated in this dividing process. This causes less undesirable conditions.

STUDIES OF AUXILIARY PASSAGES

Outlet conduits.

The outlet conduits could not be built directly through the model of the dam, because the model was located over a concrete wall which was part of the permanent building. (See fig. 333.) Instead, the water was brought under

the model through an 8-inch pipe and carried through the dam parallel to its axis. Outlet conduits were opened out of this header pipe perpendicular to the center line and carried through the downstream face of the dam. Figure 330 shows this model set-up. A single slide gate at the entrance from the forebay was the only control provided at first.

As originally installed, each of the eight outlet conduits was carried through the model with the center line at elevation 865. Visual observations and pictures were made of the action of the stilling pool with conduits wide open and the pond held at elevation 1,034, with the conduits closed and various flows over the spillway, and with all conduits open and 200,000 cubic feet per second flowing over the spillway.

With this set-up, the jets from the outlet conduits fell into the pool near the toe of the slope and caused serious disturbances. The set-up was generally very unsatisfactory, even with the addition of deflectors above the openings.

In the next tests the center line of the slide gates remained at elevation 865, and the invert of the outlet conduits changed to follow a parabola from the slide gates and connect with the tangent to the 60-foot-radius bucket at elevation 828.5. (See fig. 26.) The procedure of testing followed closely that given above except the various sills and diffuser baffles used on the apron were also tested in conjunction with the sluices to see if a more even distribution of velocities across the apron and sill could be obtained. Also, controls were made for each conduit so they could be operated singly or in combinations. The conduit was either completely opened or closed at all times, as this was the assumed method of operation expected on the prototype.

In the original design the conduits were brought through the dam in pairs. As a result, the jets impinging on the spillway pool caused unsatisfactory conditions. It was hoped to remedy these conditions by spreading the outlets more evenly across the bucket. Accordingly, four conduits were held with a center line perpendicular to the axis of the dam, and the other four (one of each pair) were diverged at an angle of 11° from the other one in the pair. (See fig. 343.) The invert was held at the same slope as before. Procedure similar to the above was followed in testing.

Tests were made on this set-up to determine the pressures existing in the sluiceways when water was flowing over the dam and not through the conduit openings. To do this, two conduits were sealed from the header and two openings inside each conduit were connected to gages. Runs were made with flows of 20,000, 40,000, 80,000, 120,000, 160,000, and 200,000 cubic feet per second and a series of observations on the gage taken for each run. The results of these tests are given in table 156.

As a means of decreasing the negative pressure found in the conduits, a parabolic hump was incorporated into the dam extending from the 0.7 slope down to the 60-foot-radius bucket through which the sluices opened. Testing procedures similar to those previously described were used for these tests. Table 156 shows the results of these tests.

Various types of hoods on the face of the dam above the conduit openings were tested on the model prior to installing the parabolic hump. Pressures shown in table 156 in the conduits were measured, as stated above, on the best form of these hoods.

The results of these tests show a comparatively wide range of pressures in the outlet passages as the quantity over the spillway is increased from very low flows to maximum discharge. The negative pressures might be useful in increasing the discharge through the conduits when they are operating in conjunction with the spillway, but at the same time if the outlet conduit control gates were closed and water was flowing over the spillway, these gates would have to be stronger to withstand the pressure. As the pressure varies for different flows, the effect could not be relied upon to assist in the flow through the conduits.

Conditions were improved by diverging the outlets, but neither the action of the water flowing over the dam and past the outlets or the general appearance of the dam was satisfactory. To correct these factors, the shape of the outlets through the face of the dam and bucket was changed.

The first change was to maintain the sides of the straight conduits parallel to their center lines. In the diverging conduits, the side farthest from the straight conduit was made parallel to the center line of the diverging conduit. The side of the conduit next to the straight one was made parallel to the center line of the straight one.

From a review of the results obtained from the outlet conduit tests and a study of the results of the pressure tests, it was decided to change the operating program for the model. This change was to allow the outlet conduits to discharge their maximum capacity as the flood approached and to remain open after the spillway came into action. At the maximum discharge of 240,000 cubic feet per second, the outlet conduits would be discharging 40,000 cubic feet per second and the spillway 200,000 cubic feet per second. This procedure would greatly reduce the probability of erosion on the floors of the outlet conduits at the openings and would eliminate the necessity of deflectors of any type over the outlet openings.

TABLE 156.—*Pressures in outlet conduits*

Prototype discharge in cubic feet per second	Model pressure from water column Ft.		Prototype pressure calculated Ft.	
	Straight outlet	Diverging outlet	Straight outlet	Diverging outlet
1	2	3	4	5
20, 000:				
a	−0. 0063	−0. 0059	−0. 454	−0. 425
b	−. 0698	−. 0888	−5. 026	−6. 394
c	−. 0286	−. 0170	−2. 059	−1. 224
40, 000:				
a	−. 0240	−. 0246	−1. 728	−1. 771
b	−. 0697	−. 1512	−5. 018	−10. 886
c	−. 0421	−. 0544	−3. 031	−3. 917
80 000:				
a	−. 1378	−. 1439	−9. 922	−10. 361
b	+. 0374	−. 1058	+2. 693	−7. 618
c	−. 0568	−. 2656	−4. 090	−19. 123
120, 000:				
a	−. 1101	−. 1169	−7. 927	−8. 417
b	+. 1134	−. 0481	+8. 165	−3. 463
c	−. 1031	−. 2826	−7. 423	−20. 347
160, 000:				
a	−. 0681	−. 0664	−4. 903	−4. 781
b	+. 3099	+. 2169	+13. 457	+5. 321
c	−. 0621	−. 2441	−4. 471	−17. 575
200, 000:				
a	+. 0119	−. 0024	+. 857	−. 173
b	+. 1869	+. 0739	+22. 313	+15. 617
c	+. 0549	−. 0796	+3. 953	−5. 731

a = 60-foot radius deflectors, each deflector covering two outlets.
b = No deflectors over outlet openings.
c = Parabolic hump over outlet openings.

Because half of the outlet conduits came through the dam perpendicular to the axis and the other four at an angle to the axis, different conditions existed in the pool when different conduits were operated singly or in combination. This necessitated the determination of an operating program for the conduits. From a series of tests in which the conduits were operated singly and in combinations of twos, threes, fours, etc., the results showed that the gates should be opened in the following order to give best flow conditions. The sluiceways are numbered from left to right looking downstream. Outlet No. 3 should be opened first, then No. 6, then No. 8, then No. 1, then No. 4, then No. 5, then No. 2, then No. 7.

This latest change showed a decided improvement, but the symmetrical appearance of the dam was disturbed. (See fig. 343.) In order to improve the appearance of the openings and further spread the discharge jets, the sides of the outlets of the straight sluices were diverged with respect to their center lines, an amount such that the width of the floor and the openings was equal to the floor width at the openings of the diverging sluiceways.

Needle valves.

In the original design of the dam, two 72-inch needle valves located in the river end of the powerhouse and discharging into the tailrace next to the spillway training wall were included. Operation disclosed immediately that the

discharge of the jets of water in the tailrace caused unsatisfactory conditions. A solution was worked out and recommended with possible modifications, but as later plans for the project eliminated the needle valves entirely, further study was unnecessary.

Powerhouse water discharge.

After the first six tests were run on the model, the powerhouse was so constructed as to simulate turbines discharging through draft tubes. The correct flow was maintained through the powerhouse and the tail water held at the height corresponding to the flow, and visual observations were made.

No quantitative studies of the flow conditions from the powerhouse were made. With the maximum flow (8,000 cubic feet per second) coming from the powerhouse, no undesirable conditions were observed with either the spillway or the outlet conduits, or both, discharging.

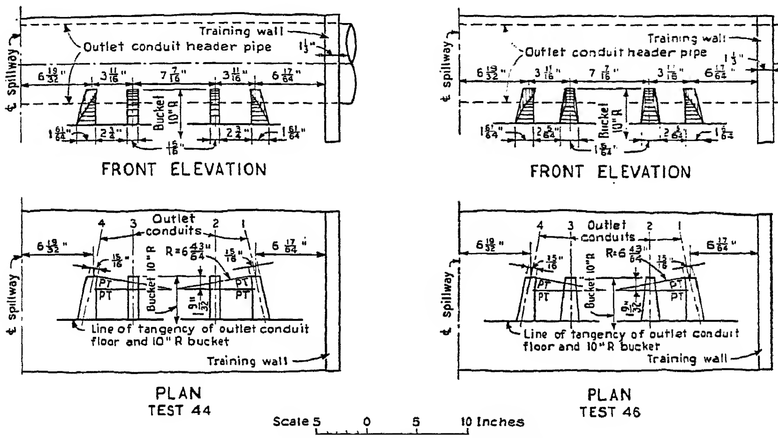


Figure 343.—*Details of outlets.*

COMPARISON OF PROTOTYPE WITH MODEL

Unusually heavy rains on the Ohio River drainage basin in December 1936 and January 1937 resulted in flood stages all along the Ohio River. During this period practically the entire flow of the Clinch and Powell Rivers above the dam was stored in the reservoir. This necessitated raising the spillway drum gates a maximum of about 11 feet above the concrete spillway crest to hold back the floodwaters. Late in January it was seen that moderate discharges would not cause further rises on the Ohio and Mississippi Rivers, and accordingly water was released to provide storage space for possible additional run-off. Much of the release was accomplished by lowering the crest gates and allowing the water to discharge over the spillway.

Stilling basin.[5]

Advantage was taken of this opportunity to observe the action of the spillway and stilling basin for heads on the gates up to 9.7 feet and for spillway discharges up to 30,000 cubic feet per second. Several special arrangements of the gates were tried out in order to compare the results with the model tests.

Observations of flow over the spillway and through the outlet conduits of the prototype lead to the following conclusions:

1. The hydraulic jump forms satisfactorily when all three spillway gates are operated alike.

[5] Hickox, G. H., Performance of Norris Stilling Basin During 1937 Flood, Tennessee Valley Authority, July 1938.

2. Operation of the stilling basin is not satisfactory when the spillway gates are not operated alike. Symmetrical arrangement of gate openings is not a sufficient condition for satisfactory operation of the stilling basin. The arrangement of openings must be uniform as well as symmetrical.

3. The stilling basin does not operate as well for outlet conduit discharge as for spillway discharge over uniformly operated gates. Whenever the desired discharge can be obtained by spillway operation alone, outlet conduit discharge should be avoided.

4. At low discharges, and with water flowing over the raised drum gates, waves exist on the face of the spillway. They are caused by vibration of the nappe as it falls over the gate to the crest of the dam. The vibration of the nappe is apparently not due to a lack of air supply beneath it. The true cause is not known.

5. When the gates are raised above the crest, air is drawn beneath the nappe. It escapes through the nappe a short distance below the crest of the dam, making the sheet of water on the face of the spillway thicker until all the air has escaped.

6. When the gates are lowered to the crest, entrainment of air at the surface of the water appears to be governed by the depth of water over the crest. The greater the depth, the greater is the distance traveled before entrainment of air begins. Since both the depth of flow and the distance traveled are greater, the velocity must also be greater. It seems probable that depth of flow, as well as velocity, is a factor governing the entrainment of air.

7. Performance of the prototype agrees well with the results of model tests for all conditions observed. Observations of stilling-basin operation for much higher discharges are desirable in order that the prediction of the model may be verified over as great a range of conditions as possible.

Comparison of the operation of the prototype with the model, while not exact, owing to the fact that similar conditions were not obtained, nevertheless offers good enough agreement to show the value of model tests. In all cases the model predicted faithfully the salient characteristics of flow and the general type of operation of the stilling basin.

ARCHITECTURAL STUDIES

The only purely architectural studies made were on the spillway bridge and piers. The original bridge was designed as a through-truss type in 100-foot spans from pier to pier. This was replaced by a pony-type truss and finally by a plate girder bridge. The latter necessitated changes in pier design, which, when tested, produced no undesirable results. In the final set-up, elevator towers (one of which was later eliminated) at each end of the spillway were built.

The results of the architectural studies in connection with the sluice openings are shown in figure 344. The appearance of the completed project was kept constantly in mind in conjunction with all changes. Both the through-truss type and the pony-truss type bridge appeared to be somewhat out of proportion when placed across the spillway. The plate girder bridge appeared to be much more in keeping with the other features of the dam and added greatly to the appearance of the structure. The change in design of the downstream face of the piers to support this bridge was beneficial to the aeration of the jets from the gates when they were near to and at their maximum elevation. Clearance beneath the structure was as good as in previous designs.

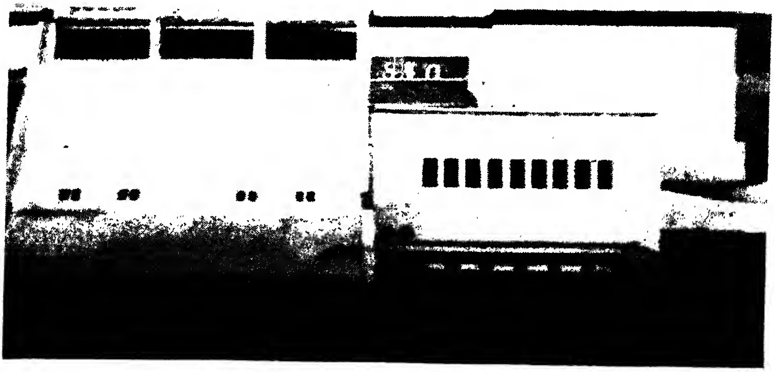
STRUCTURAL MODEL STUDIES

In connection with the design of the dam, it was deemed advisable to check at least part of the mathematical analyses by use of a structural model. The model permitted not only a check of some of the mathematical analyses but also the observation of some deflections and stresses which would not have been obtainable by other means.

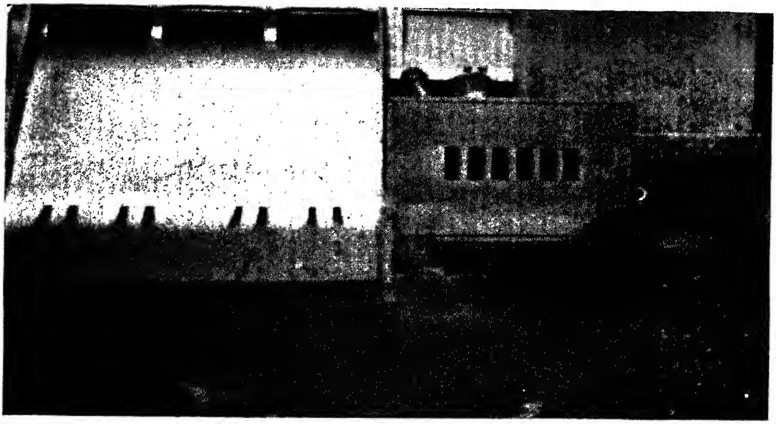
The United States Bureau of Reclamation, which conducted the tests [6] for the Authority, had previously conducted similar tests and was well equipped with both experienced personnel and proper equipment with which to conduct

[6] Smith, E. D., Tests on Plaster-celite Model of Abutment Section of Norris Dam, U. S. Bureau of Reclamation Technical Memorandum No. 433, January 31, 1935.

the Norris Dam tests. The work on the Norris model was a continuation of the testing methods applied to a model of Grand Coulee Dam[7] incorporating changes and improvements that the previous work indicated to be desirable; particularly, these changes included increased depth and the extent of foundation, provision for discontinuities in the foundation, and means for measuring stress concentration at the corners.



(A) Original design.



(B) Final design—Outlet conduits.

FIGURE 344.—*Results of architectural studies.*

The first work of importance on model studies of this type for gravity dams was that of Wilson and Gore.[8] The model arrangement used in the Norris tests

[7] Smith, E. D., Model Tests of Grand Coulee Dam, U. S. Bureau of Reclamation Technical Memorandum No. 372, March 1, 1934.

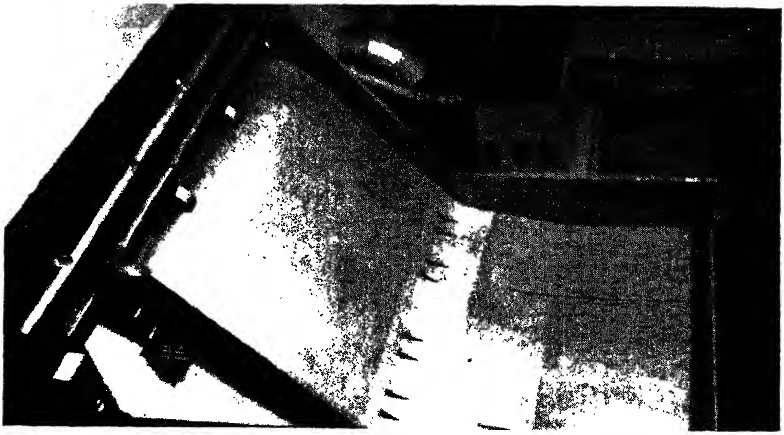
[8] Wilson, John S., and Gore, William; Stresses in Dams: An Investigation by Means of India Rubber Models. Minutes of the Proceedings of the British Institute of Civil Engineering, vol. 172, pp. 107–133, 1907–8.

is similar to that used by Wilson and Gore, except that in the case of Norris the model was much larger, a liquid live loading was used, and more sensitive apparatus was employed for the measurement of strains and deflections.

Model material.

A mixture of commercial building plaster and celite, a silica product commonly used as an admixture to concrete, was used for the construction of the model. The purpose of the celite was to dilute the plaster to produce a lower modulus of elasticity.[9]

The proportions of the material used in the model were 1 part celite, 2 parts plaster, and 3¾ parts water, all by weight. This mixture ordinarily sets in about half an hour when mixed without the addition of retarders or accelerators. Where setting is too fast, the speed can be retarded by the addition of a small amount of aqua-gel. A very desirable feature of this mixture is that it sets at practically constant volume. When freshly mixed, the resulting product has a consistency of pancake batter and will flow easily and completely



(C) Final design—Spillway bridge and piers.

FIGURE 344.—*Results of architectural studies.*

fill a form to any shape likely to be used. Some agitation is required to expel air bubbles that have become entrained in the mix. The material remains plastic even after setting until thoroughly dried. The drying period is usually about 10 days for a 3-inch thickness. When thoroughly dry, this mixture forms a homogeneous, isotropic, and highly elastic material. The average values of modulus of elasticity and Poisson's ratio determined from test specimens were 120,000 pounds per square inch and 0.2, respectively.

Similarity conditions.

When comparing the behavior of the model with the prototype, it is necessary that the following relationships be known:

1. Scale—$\frac{1}{n}$

2. Specific gravity—$\frac{1}{G}$

3. Ratio of moduli of elasticity of model and prototype—$\frac{E_m}{E_p}$

[9] Simmonds, A. W., Construction of Plaster and Celite Model of Hoover Dam, U. S. Bureau of Reclamation Technical Memorandum No. 306, October 1, 1932.

The model was loaded with mercury and the dead weight of the dam was produced by weight hung on the model to balance the liquid pressure in the same proportion that concrete would balance a water load.

With a model scale equal to $\frac{1}{n}$ and a specific gravity loading equal to G times that of water, the unit liquid pressure at the base of the dam will be $\frac{n}{G}$ times that of the model, and this ratio will be the same for the stresses. The model scale selected was $\frac{1}{60}$ and G equal to 13.6; therefore, the stresses of the dam would be $\frac{60}{13.6}=4.41$, or the stresses in the dam would be 4.41 times those of the model. If the modulus of elasticity of the concrete in the dam is equal to E_d, and the modulus of elasticity of the model is equal to E_m, the ratio between unit strain in the model and in the dam will be the ratio of the unit stress multiplied by the ratio of the two moduli of elasticity or $\frac{n}{G}\times\frac{E_m}{E_d}$. These strains are acting over lengths of the dam n times the corresponding length for the model. The total deformation of the dam given as the deflection at a certain point will therefore be n times the ratio for the unit strain compared to the total deformation of the model. The ratio between the deflection of the dam and the deflection of the model will be $\frac{n^2E_m}{GE_d}$. Assuming a modulus of elasticity for concrete equal to 3,000,000, this ratio will be $\frac{60\times 60\times 120{,}000}{13.6\times 3{,}000{,}000}=10.59$, or the deflection of the dam will be 10.59 times the deflection of the model. This latter comparison can be made only between the model slab and a similar slab cut through the dam. If there is a difference in the modulus of elasticity of the concrete of the dam and the rock foundation, the deflections will be altered. Also, in the actual dam, the lateral restraint of the sections, due to the continuity of the structure, will reduce the deflection.

Stress-strain relations.

As stated before, the stresses in the model and prototype are connected by the relation of scale and specific gravity, but the strain relationship is not so simple. The model slab, having no lateral forces acting, is under condition of plane stress, while in the dam the lateral forces between sections prevent lateral strain so that a section through the dam will also be under plane strain. However, since the lateral stresses cannot affect the stress in the plane normal to them, the simple stress ratio between model and prototype is not affected. In reducing the strain measurements on the model to stresses, a stress-strain relationship was developed.[10]

NOTATION

ϵx, ϵy, Maximum and minimum principal strains, respectively.

ϵH, ϵD, ϵV, Measured strains about a point on gage lines making angles of 0°, 45°, and 90° with the horizontal.

σx, σy, Maximum and minimum principal stresses, respectively.

σH, σV, Horizontal and vertical stress components, respectively.

μ=Poisson's ratio.

θ=The angle between ϵx principal strain and the horizontal.

The three following equations give the magnitude and direction of the principal strains in terms of the measured strains:

1. $\epsilon x+\epsilon y=\epsilon H+\epsilon V$
2. $\epsilon x-\epsilon y=(\epsilon H-\epsilon V)$ Sec 2θ
3. Tan $2\phi=\dfrac{2\epsilon D-\epsilon H-\epsilon V}{\epsilon H-\epsilon V}$

The principal stresses accompanying these principal strains are given by these equations:

$$\sigma x=\frac{E}{1-\mu^2}(\epsilon x+\mu\epsilon y)\,;\ \sigma y=\frac{E}{1-\mu^2}(\epsilon y+\mu\epsilon x)$$

[10] Smith, E. D., Model Tests of Grand Coulee Dam, U. S. Bureau of Reclamation Technical Memorandum No. 372. March 1, 1934.

From which it follows that the horizontal and vertical components will be:

$$\sigma V=\frac{E}{1-\mu^2}(\epsilon V+\mu\epsilon H);\quad \sigma H=\frac{E}{1-\mu^2}(\epsilon H+\mu\epsilon V)$$

Horizontal shearing stress is given by the following relation:

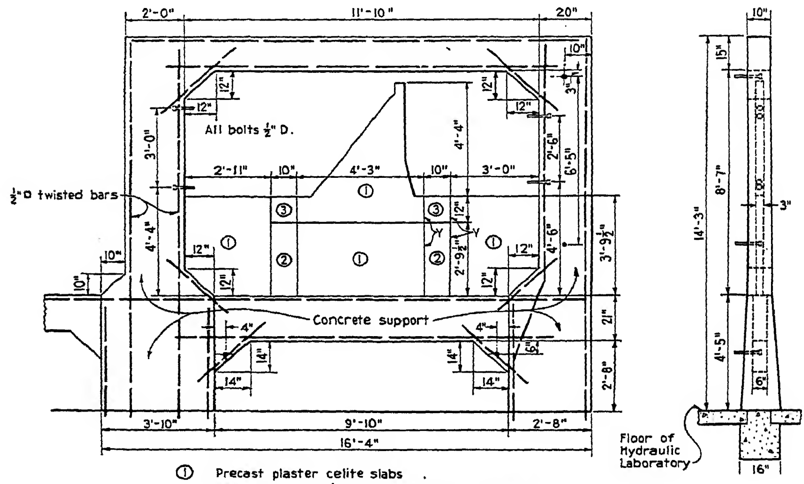
$$\tau=\tfrac{1}{2}(\sigma x-\sigma y)\sin 2\theta$$

where again θ is the angle of ϵx with the horizontal.

Design of model.

It is desirable to have a model of this type as large as space would permit since with a given specific gravity the intensity of strain increases in proportion to the linear dimensions, and because large strains can be measured with greater accuracy than small strains.

In this model the supporting frame was limited to an over-all height of 14 feet. The dam, as designed at the typical abutment section used in these tests, had a total height of 260 feet, so that with a scale of 1:60, the model height



① Precast plaster celite slabs

② Sections poured with foundation slabs in place

③ Sections poured with dam section in place

All joints shellacked to insure good bond except joints Y-Y which were greased so as not to take tension

FIGURE 345.—*Model and supporting frame.*

was 52 inches. With a foundation depth of 48 inches, the total height of the model and foundation was 8 feet 4 inches. This left 7 feet 8 inches of the available space under the model for the supporting frame and loading mechanism.

The frame for this model was designed to be much heavier than the strength requirements demanded in order to insure that the frame would remain rigid.

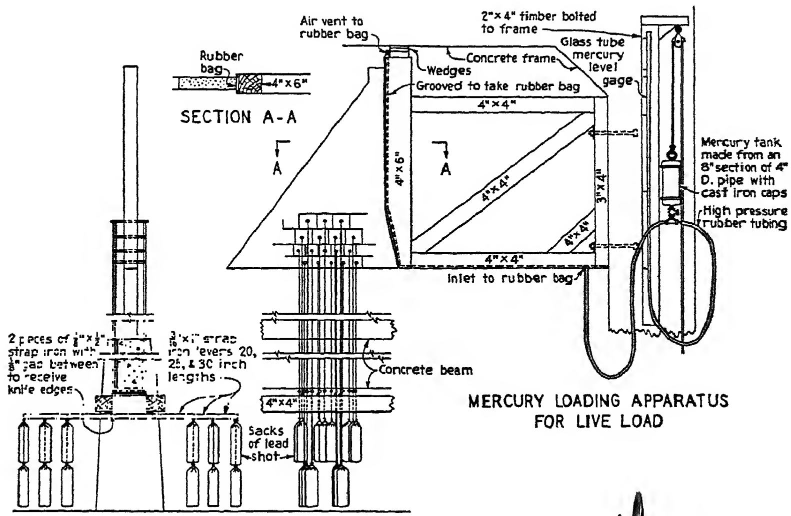
Figure 345 shows the design of the reinforced concrete supporting frame and the model slab. Because of its large size it was necessary to cast the model slab in sections which could be assembled in the supporting frame.

The dead weights of the model were produced by the use of external loads to give an equivalent unit weight of 13.6×150, or 2,040 pounds per cubic foot. With a 3-inch thickness of model slab, this gives 3.54 pounds per square inch of model section. The dead load was distributed through the dam section by dividing the surface of the dam section into a large number of subdivisions, and through the centers of gravity of these subdivisions steel rods were placed from

which the load could be applied. The amount of load to be applied to each subdivision was equivalent to the volume of the subdivision multiplied by the unit weight. It was not possible to use a uniform size of subdivision because of the necessity of preventing the loading levers from overlapping.

In the upstream region of the dam, a large number of loading points occur very close together horizontally, so it was necessary to use three lever arm lengths in order that the different weights would not interfere with each other.

The live load system included means for applying hydrostatic pressure to both the upstream and downstream faces, corresponding to the reservoir pressure and tail water pressure, respectively. The apparatus for applying these pressures consisted of wooden frames which held rubber bags of the same width as the model slab against the faces of the dam and along the top of the foundation. The rubber bags were connected by high pressure hose to steel tanks which were filled with mercury. By hoisting the tanks to various heights, the mercury in the bags could be adjusted to any desired elevation.



TYPICAL ARRANGEMENT OF DEAD LOADS ON LEVERS

FIGURE 346.—*Arrangement for applying dead and liv[illegible]ds.*

Construction of model.

After the concrete supporting frame was constructed, t[illegible]ctions of the model and the foundation were assembled in the order [illegible]n figure 345. The three lower sections, marked (1), were set on a l[illegible]resh quickset plaster to form a bond with the frame. The sections [illegible]2) were then poured in place to join the first sections together. Th[illegible]tion, marked (1), was cast to approximate shape and placed on the [illegible]ection of the base on a freshly mixed layer of plaster celite of the s[illegible]ortions as the model slab. This gave a bond between the slabs withou[illegible] discontinuity in the elastic properties. Finally the closing plugs, ma[illegible]ere poured in p[illegible]ce to complete the slab. The surface of the upstrea[illegible]een the dam s[illegible]tion and closing plug (3) was formed by a sheet [illegible]per so that h[illegible]zontal tensile stresses could not be transmitted th[illegible]ndation.

[illegible]en the model slab was completed, the correct [illegible] was cut [illegible] saw. The entire slab was then given tw[illegible]ne and two [illegible] paint to prevent the moisture conten[illegible]and to pro- [illegible]m ab[illegible]

The surface of the dam was marked off into subdivisions and the centers of the subdivisions were drilled to receive steel pins from which the dead load weights were suspended. Below each steel pin in the model was a similar pin under the concrete beam, and the ends of the two pins were connected by piano wire. The center of the lower pin was connected by a wire to a lever through which the loads were applied. In this manner the dead loads were symmetrically applied to the model slabs.

The live load apparatus was essembled in such a manner as to permit variations in tail water and headwater elevations.

Optical strain gages were used to measure the strain on the face of the dam section. Strain gage pins were located in sets of four on the circumference of 2-inch circles. They were arranged with sets of horizontal and vertical gage lines alternating with sets of 45° and 135° diagonals.

Boundary strains at the corners where the dam joined the foundation were measured with Huggenberger tensometers. The strain pins for these gages were made from lath nails forced into small drilled holes in the model.

For the measurement of deflection, "Last Word" dials and Ames dials reading to 0.0001 inch were used. These were supported on two invar steel rods, one attached to the model and the other attached to a reference member. The dials were held rigidly to one rod and measured the amount of displacement of the other. The vertical deflections were measured with reference to the concrete beam at the bottom of the model, while the horizontal deflections were measured with reference to a wooden frame supported by this beam. A dial was set at the top of this frame to bear against the bottom cord of a steel roof truss to check on the movements of the reference frame.

Testing procedure.

Tests were run on this model for deflections and strains due to dead load only and for dead load combined with full reservoir and tail water pressures. A complete testing cycle consisted of the following operations:

1. Setting and adjusting the dials and strain gages for the points being observed.
2. Making no-load observations.
3. Applying dead load.
4. Making dead load observations.
5. Applying live load to model.
6. Making combined load observations.
7. Releasing all loads.

For each arrangement of instruments, the above cycle was run a sufficient number of times to give good average values. To facilitate applying and releasing the dead load, three wooden beams were placed on each side of the concrete frames. The loading weights were tied to these beams as well as to the iron levers. When the ends of the wooden beams were lifted, the weights of the dead loads were transferred to the beams. When the wooden beams were lowered, the dead loads were transferred back to the levers.

Where horizontal deflections were measured between the model and the wooden frames, a small vertical deflection of the bottom concrete beam caused the horizontal movement of the top of the reference frame. The amount of this horizontal movement was registered on a dial bearing on the roof truss of the building. In order to eliminate this movement, a screw jack was placed under the concrete beam. When the live load was applied, the dial at the top of the reference frame was read. The jack was then raised until the concrete beam was pushed back to its original position. When this position was reached, the dial at the top of the reference frame indicated zero deflection. The horizontal deflections of the model could then be read and recorded without further correction.

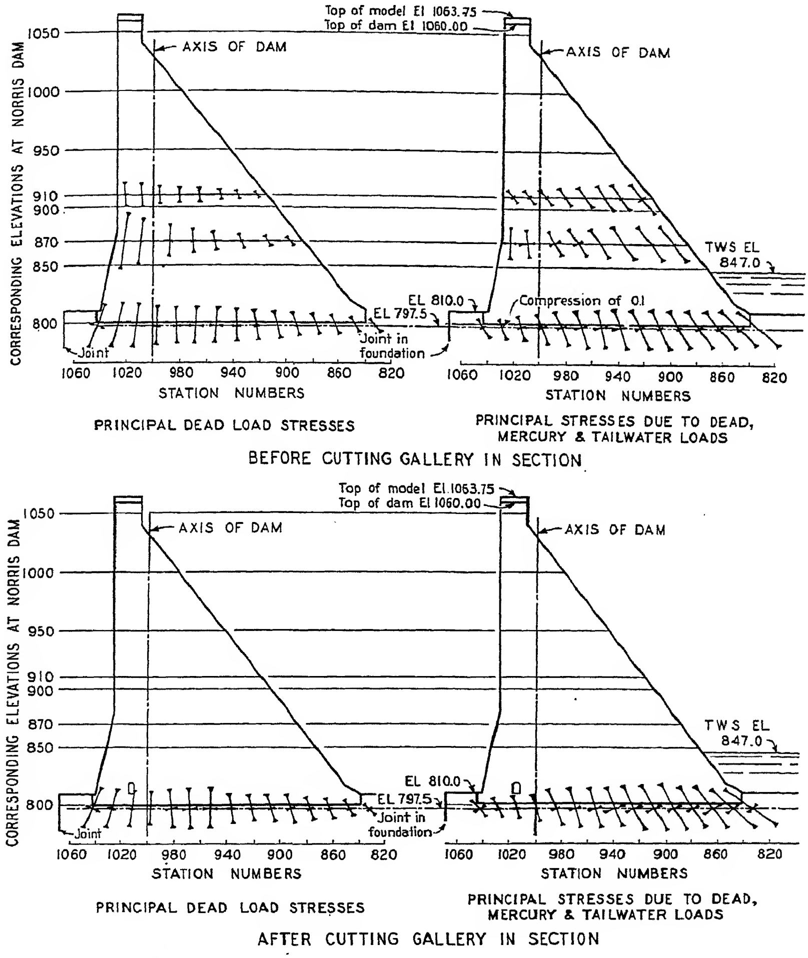
Strain measurements.

The strain measurements on the face of the model section and foundation were made with a Tuckerman optical strain gage.[11] The smallest displacement that can be measured with this apparatus is $\frac{1}{2000000}$ of an inch per inch.

For the computation of the principal strain at a certain point, it is necessary that the strains on three different directions be known. If only one point were

[11] Tuckerman, Dr. L. B., American Standards for Testing Materials, Proceedings, vol. 23, 1923, p. 602.

being investigated, a rosette of six gage points on a circle about the point would be required. It was found that this number of rigid plugs set in a soft material caused a reduction in the deformation inside the rosette of gage points. Where strains were being measured at several points in a row, this difficulty was eliminated by setting the horizontal and vertical rosettes alternately with the



compression
tension
Station 1000 coincides with axis of model.
Stresses were computed from strains measured with Tuckerman optical strain gages.
Conversion factor for stress in dam = $\frac{60}{13.6}$ = 4.412 x stress in model.
Foundation joint at sta 1067.5 was open at the time of these tests.

Scale 100 0 100 200 300 Lbs per sq in.
Stress in dam

Scale 20 0 40 80 Lbs per sq in.
Stress in model

Figure 347.—*Principal stresses—Dead and live loads.*

45° and 135° diagonals. In this way only four strain gage pins were set in each rosette. In arranging the strains for computation, the gage readings were plotted as ordinates to a common horizontal scale. A smooth curve was drawn through the plotted point for each set of readings: the horizontals, verticals, and each of the diagonals. Any vertical line drawn through these curves will then give the four strain components for the points from which the vertical line was drawn. A check upon the accuracy of the observed readings comes from the relation that the sum of two strains at right angles to each other at a point is a constant quantity. Therefore, the sum of the horizontal and vertical components must be equal to the sum of the diagonal components. Although two different diagonal strains were observed, only the 45° diagonal was used in the computation of principal strains and stresses.

TABLE 157.—*Maximum vertical and horizontal stresses computed from strain measurements*

Elevation	Maximum vertical stress						Maximum horizontal stress					
	Dead load			Dead load+LL			Dead load			Dead load+LL		
	Station No.	Model	Dam	Station No.	Model	Dam	Station No.	Model	Dam	Station No.	Model	Dam
910	1,020.0	−21.6	−95.3	937.5	−18.8	−82.9	1,020.0	2.9	12.8	980.0	−7.5	−33.1
							922.5	−0.4	−1.8	-------	------	-------
870	1,021.25	−47.8	−210.9	1,021.25	−27.3	−120.4	988.25	−3.6	−15.9	988.25	−11.7	−51.6
							1,021.25	1.0	4.4	-------	------	-------
797	1,027.5	−37.4	−165.0	937.5	−33.4	−147.4	967.5	−7.8	−34.4	832.5	−24.2	−106.8
							1,042.5	4.5	19.9	-------	------	-------
797[1]	967.5	−33.7	−148.7	892.5	−38.4	−169.4	967.5	−10.6	−14.6	832.5	−35.6	−157.1
							1,042.5	3.3	-------	-------	------	-------

[1] Stresses after grouting gallery had been cut in model.

Stresses are in pounds per square inch.
— denotes compression.
LL denotes live load.

The maximum value of stresses computed from the strain measurements are given in tables 157 to 159, inclusive. A maximum prototype vertical compressive stress of 210.9 pounds per square inch occurred at elevation 870 at the upstream face and was due to dead load only. No tension in the vertical direction was found. A maximum horizontal compressive stress of 157.1 pounds per square inch occurred at elevation 797 at the downstream toe of the dam. This stress was measured after the grouting gallery had been cut in the section. Small horizontal tensile stresses in the model were found due to dead loads. The maximum was 19.9 pounds per square inch at elevation 797. It was measured before the grouting gallery was cut in the section. No horizontal tension was found due to the combined dead and live loads.

TABLE 158.—*Horizontal shearing stresses computed from strain measurements*

Elevation	Maximum horizontal shear					
	Dead load			Dead load+LL		
	Station No.	Model	Dam	Station No.	Model	Dam
910	922.5	2.0	8.8	922.5	16.5	72.8
870	1,021.25	6.4	28.2	897.5	18.8	82.9
797	1,042.5	14.7	64.9	832.5	21.4	94.4
797[1]	1,042.5	16.7	73.7	832.5	22.9	101.0

[1] Stresses after grouting gallery had been cut in model.

LL denotes live load.
Stresses are in pounds per square inch.

The maximum horizantal shearing stress was at elevation 797 at the downstream toe and was due to the combined dead and live loads. Before cutting the gallery, this stress was 94.4 pounds per square inch, and after cutting the gallery the stress was increased to 101 pounds per square inch.

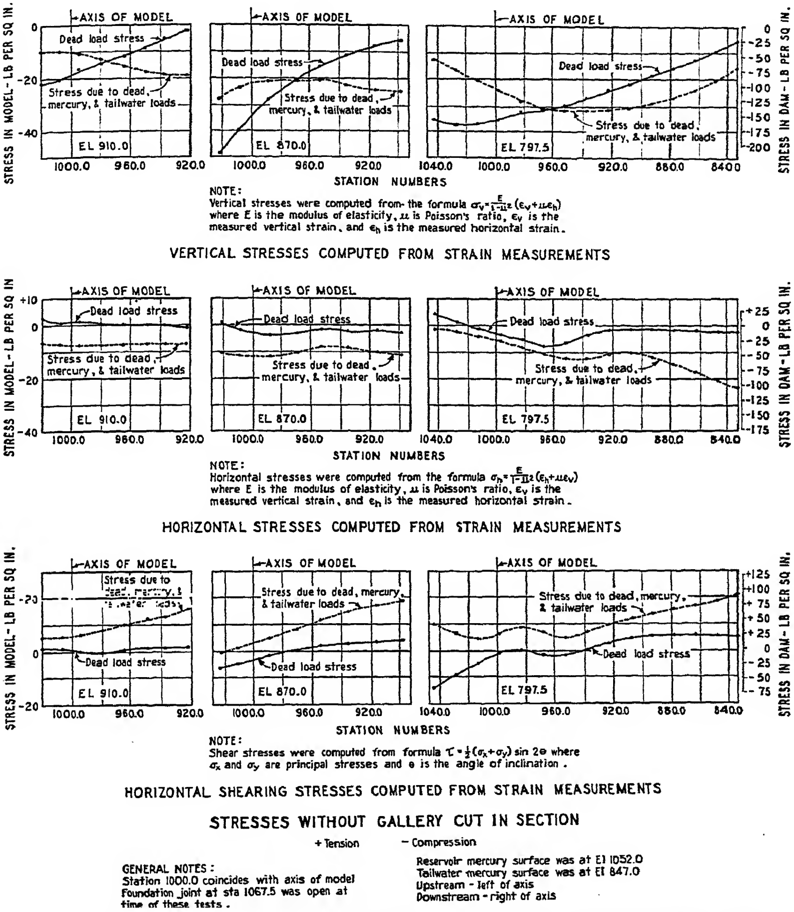


FIGURE 348.—*Vertical and horizontal stresses and horizontal shearing stresses.*

Maximum principal stresses are recorded in table 159. The largest of these occurred at elevation 797 after the gallery was cut and was due to dead load. The maximum compressive stress was 233.4 pounds per square inch and occurred at the downstream toe. The maximum tension occurred at the same elevation but nearer the center of the base and was 43.7 pounds per square inch.

A comparison of the stresses obtained analytically and experimentally is given in table 160. The variation in the two sets of stresses is due to the fact that the analytical stresses were based on the straight-line distribution of stress;

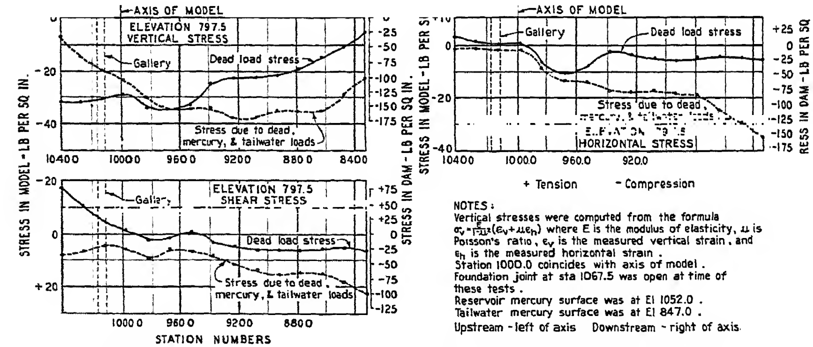
the experimental stresses were based on strain measurements of the model. The reservoir water surface elevation was 1,047 for the analytical stresses and 1,052 for the experimental. The highest stresses obtained analytically were those which occur parallel to the faces of the dam. These are compared with the larger principal stresses that occur nearest the point used in the computation.

TABLE 159.—*Maximum principal stresses computed from strain measurements*

Elevation	Maximum principal stress											
	Dead load			Dead load+LL			Dead load			Dead load+LL		
	Station No.	Model	Dam	Station No.	Model	Dam	Station No.	Model	Dam	Station No.	Model	Dam
910	1,020.0	3.0	13.2	922.5	4.6	20.3	1,020.0	−21.8	−96.2	922.0	−30.4	−134.1
				1,020.0	−2.4	−10.6						
870	988.25	−3.5	−15.4	1,021.25	−10.5	−46.3	1,021.25	−48.8	−215.3	897.5	−38.1	−168.1
	1,021.25	2.0	8.8	897.5	2.1	9.3						
797	1,042.5	9.4	41.5	952.5	−43.3	−191.0	1,042.5	−39.9	−176.0	832.5	−43.3	−191.0
	967.5	−7.8	−34.4	1,042.5	4.2	18.5						
797[1]	1,042.5	10.0	44.1	832.5	−52.9	−233.4	1,042.5	−38.5	−169.9	892.5	−47.0	−207.4
	967.5	−9.9	−43.7	1,042.5	4.6	20.3						

[1] Stresses after grouting gallery had been cut in model.

Stresses are in pounds per square inch.
— denotes compression
LL denotes live load



VERTICAL HORIZONTAL. AND SHEAR STRESSES COMPUTED FROM STRAIN MEASUREMENTS
STRESSES WITH GALLERY CUT IN SECTION

FIGURE 349.—*Vertical, horizontal, and shear stresses (with gallery cut in section).*

The results shown in table 160 show that the stresses obtained by assuming a straight-line variation of stress are larger than those that are based on a nonlinear variation of stress.

The figures for the observed stress in table 160, giving the comparison between observed and computed stresses, are taken from measurements made in the interior of the model. The stress diagrams were extended to the faces of the dam to obtain the stresses at these points. At the corners there were stress concentrations of which a special study was made. A Huggenberger tensometer was used to measure the strains occurring at these locations. These instruments were set on the model when the first loading was applied in order

to determine if any plastic flow occurred due to the concentration. It was found that at the downstream corner adjoining the foundation where the greater strain occurred the first load gave strains about 50 percent higher than the strains obtained after a considerable number of tests had been run. The stresses are shown in figure 350.

TABLE 160.—*Comparison of stresses obtained analytically and experimentally*

Elevation	Vertical stress		Stress parallel to face, analytical	Maximum principal stress, experimental
	Analytical	Experimental		
DEAD LOAD STRESS AT UPSTREAM FACE				
910		95		121
900	179		179	
870		211		215
850	211		219	
800	233		243	
797		165		176
DEAD LOAD STRESS AT DOWNSTREAM FACE				
910		10		15
900	1(T)		1.6(T)	
870		25		38
850	12		18	
800	33		49	
797		24		44
COMBINED WATER LOAD AND DEAD LOAD STRESSES AT UPSTREAM FACE				
910		43		62
900	80		80	
870		120		120
850	88		91	
800	99		103	
797		56		81
COMBINED WATER LOAD AND DEAD LOAD STRESSES AT DOWNSTREAM FACE				
910		83		134
900	98		146	
870		110		168
850	140		209	
800	183		273	
797		85		191

Stresses are in pounds per square inch.
Analytical stresses are taken from figure 321.
Experimental stresses are obtained from strain measurements on model.

At the downstream toe two corners were formed by a fillet. The stress at the upper corner, due to combined live and dead loads, was —66 pounds per square inch in the first tests and —56 pounds per square inch in the later tests. In the fillet itself, the stress was —31.9 pounds per square inch in the first tests and —64 pounds per square inch in the later tests. In the same order, at the lower corner, the stresses were —104 pounds per square inch and —84 pounds per square inch. This shows that as the number of loadings increased there was a redistribution of stress. The stress at the corners decreased due to plastic flow and was accompanied by an increase in the stress along the face of the fillet. The maximum stress at the corner where the dam joins the foundation, predicted from the model measurements, was 460 pounds per square inch against 233 pounds per square inch for the interior of the dam.

It appears from these tests that if a fillet is to be used at the toe of the dam, a smooth curve transition from the dam to the foundation would be more effective in reducing stress concentration than the wedge-shaped fillet.

The Huggenberger tensometer was also used to measure stresses around the grouting gallery. Due to the small size of the gallery compared to the gage lengths used, the results were not very conclusive. Later, however, separate large-scale experiments were made to determine gallery stresses.[12]

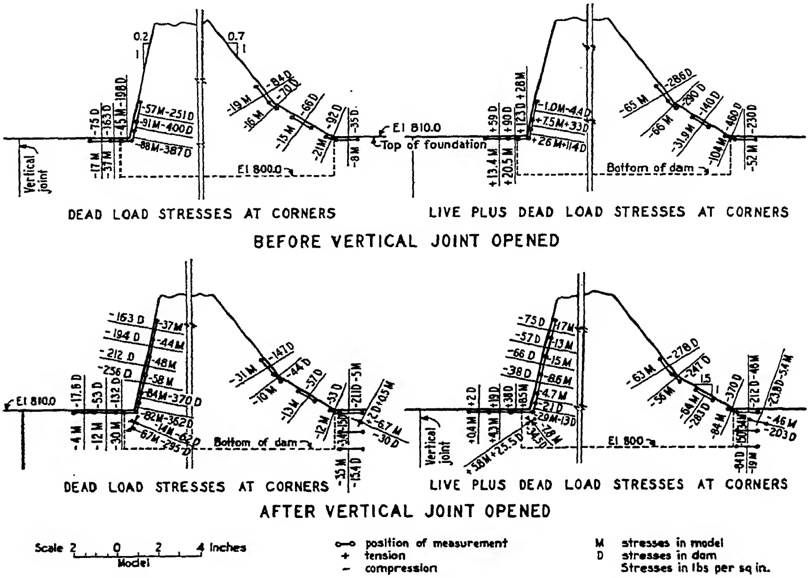


FIGURE 350.—*Dead and live load stresses at corners.*

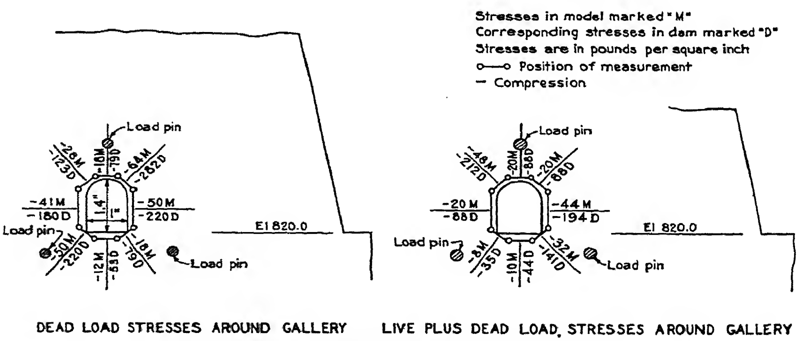


FIGURE 351.—*Observed stresses around gallery.*

Deflection measurements.

The deflection measurements were made to obtain an approximate picture of the deflection of the dam and the foundation due to the weight of the dam and to the weight of the dam plus the water load. Unlike the stresses, the

[12] Brahtz, J. H. A., Photoelastic Investigation of Stresses Around Galleries, U S. Bureau of Reclamation Technical Memorandum No. 439. April 1, 1935.

deflections of the model cannot be converted directly into deflections of the dam for reasons enumerated previously under similar conditions. They can only be used as a qualitative measure of the deflections likely to occur.

Horizontal and vertical components of deflections were measured for both upstream and downstream faces of the model and at the top of the foundation. Vertical displacements were measured in the lower part of the foundation. Figure 352 shows the results of the deflection measurements.

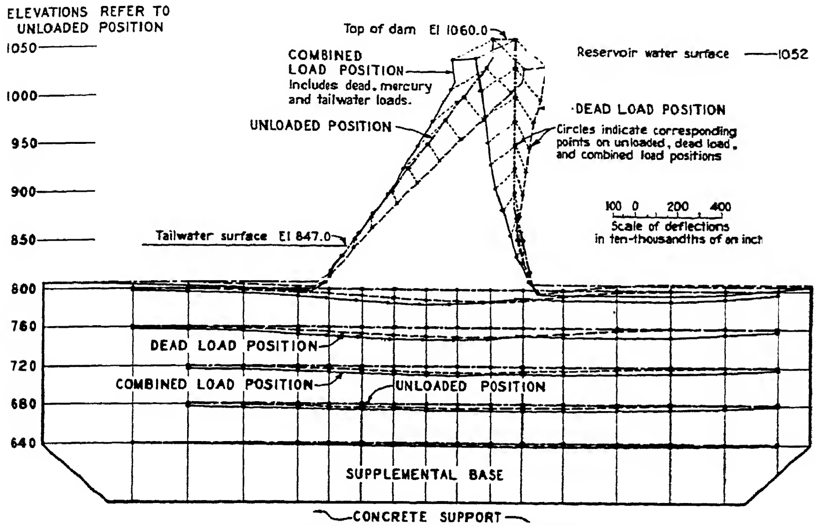


FIGURE 352.—*Deflection measurements.*

Discussion.

The theoretical accuracy of this type of model tests rests upon the application of the principle of Saint Venant. This principle states that if forces acting on a portion of an elastic body are replaced by another statically equivalent system of forces acting on the same portion of the surface, this redistribution of loading produces substantial changes in the stresses locally, but has negligible effect on the stresses at distances which are large in comparison with the linear dimensions of the surface on which the forces are changed. This principle applies directly to this type of model test. If a vertical slice is cut through a dam and its foundation, the stresses produced in the foundation by the dam will have a distribution to infinity in the foundation slab. If, then, at a finite distance from the dam, the original stresses are replaced by the reactions on a rigid supporting member, the stresses along this boundary will have undergone a change, but the statical equilibrium will remain unchanged. The more remote this supporting member is from the dam, the less will be the effect of the replacement of the original stresses by the reactions on the supporting member. In this study, the dimensions of the foundation were arbitrarily selected so that the least distance from the base of the dam to the foundation was 48 inches against a height of dam of 52 inches. Whether this distance was sufficient to eliminate the effect of the rigid support is not definitely known, but later mathematical investigations have shown that the error introduced in the region near the base of the dam was small.

There are several advantages of this type of tests over tests which require a complete model, of which the most important is the provision for obtaining dead load effects. It is possible also to make the following checks upon the results:

1. Check upon the observed strains: The sum of two diagonal strains must be equal to the sum of the horizontal and vertical strains at a given point.

2. Check upon the computed stress: The area under the stress diagram must be equal to the sum of the vertical forces.

3. Check upon the stress distribution: The center of gravity of the area under the vertical stress diagram must coincide with the center of gravity of the vertical forces.

The action of the dam section in these tests would be quite similar to a section of an actual dam where the vertical construction joints were not grouted. Where the joints are grouted so that the structure behaves as a monolith, the torsional resistance between adjacent sections produces torsional shearing strain and stresses which may affect the stresses as predicted from the model tests.

In all studies of this type, nonlinear stress distribution is observed in the lower portion of the dam, while in the upper portion the stresses approach a linear distribution quite closely. This is in agreement with the latest mathematical analysis of cantilever sections as well as with photoelastic studies.

Mathematical studies of nonlinear stress distribution and stresses at corners are based on the assumption of perfectly elastic materials; whereas, actual materials are elastic only within a certain range of stress and are subject to some plastic deformation during strain. This fact may explain why the theoretically high concentrations of stress at the corners are not observed in the model tests. When high concentrations occur, the plastic deformation will allow a redistribution of stress to occur in the model as it probably does in the actual structure. Mathematical analyses do not give the stresses where plastic flow causes a redistribution of stress.

Tractor gate model studies and tests

Model studies were made by the United States Bureau of Reclamation to determine mechanical and hydraulic characteristics of large control gates.[13] These studies were made in regard to gate design to determine hoist requirements and pressures exerted on gates due to hydrodynamic conditions. Among the various types studied was a tractor gate to be used in the Norris Dam power penstocks. After the dam had been completed, tests[14] were conducted on the prototype gate for use in checking the downpull when the gate was being operated under normal and emergency plant conditions. This section of the appendix will summarize the model studies as well as a discussion of the results of the tests made on the prototype.

Forces considered in design.

The general design of a gate hoist provides for sufficient capacity to close or open the gate under full operating head. Usually the required force to operate a gate is computed from the sum of the frictional resistances of the gate plus the weight of the moving parts for lifting the gate, and minus that weight for lowering the gate. The weight of the moving parts in water is equal to the dead weight minus the effect of buoyancy or uplift of the water. Frictional resistance varies for the different type gates. For sliding gates this force is equal to from 0.3 to 0.75 depending on the material in the leaf and seat and the water load on the leaf. In the case of fixed wheel gates the frictional resistance is equal to the axle friction plus the rolling friction of the wheel, plus a resistance due to the seal. If roller trains are used in place of wheels, the axle friction is eliminated but a link friction must be added.

There are also hydraulic forces present which may increase the load on the hoist to a point where the above design, even with the customary factor of safety, is no longer safe. This force is caused by the unbalanced water pressures on the gate leaf when it is in a throttling position. The water passing below a flat-bottom leaf exerts no pressure and may even create a partial vacuum on the under side of the leaf. As there is a pressure above the leaf—water pressure if the leaf is submerged or atmospheric pressure if it has a free-water surface—the downward force is equal to the difference of the pressures above and below the gate multiplied by the area of the bottom of the leaf. By changing the shape of the bottom of the gate or of the approach, a pressure may be built up on the bottom of the leaf which will partially or completely eliminate the unbalanced condition.

[13] Noonun, N., and Hornsby, G. J., Model Studies of the Mechanics and Hydraulics of Large Control Gates, U. S. Bureau of Reclamation, June 17, 1937.

[14] Noonun, N., Field Tests on Downpull on Norris Tractor Gate and a Comparison of Calculated Loads, U. S. Bureau of Reclamation, March 9, 1938.

Description of tractor gate.

The tractor gate developed to meet set requirements for an emergency gate for the Norris power units closes an opening 16.5 by 28.5 feet in size under a maximum head to the center line of the gate of 176 feet. It was desired that the gate and hoist be of such design that the gate, when not in use, could be raised above the water surface for storage and maintenance. This practically eliminated the use of a stem hoist. As a rope hoist cannot exert a downward thrust, the gate must be completely lowered by its weight alone. To do this the friction must be reduced to a minimum. This was accomplished by means of roller trains, the frictional resistance of the rollers being only about 0.01 of that caused by sliding surfaces. Due to the fact that the gate must be lowered by its weight in water, it was necessary to place the skin plate

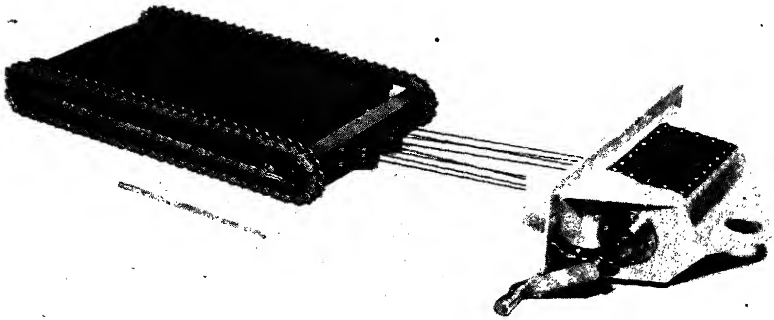


FIGURE 353.—*Tractor gate model.*

on the downstream face, allowing the water to fill completely the leaf section.

The next problem to be considered was the sealing of the gate under the 176-foot head. The common flexible metal or rubber seal could not be used because of the high head and because of the drag offered by such a seal during the raising or lowering of the gate. Sealing by a metal-to-metal contact similar to the simple slide gates was deemed a desirable feature, and a movable wedge mounted on rollers was incorporated in the roller train, the wedge to be actuated by a toggle mechanism of such a design as to insure proper sequence of all movements in relation to the gate.

Starting from the above water surface position, the sequence of the movements of the gate is as follows:

1. The leaf is lowered on the roller trains until it is directly in front of the frame opening.
2. At this point the leaf rests on stops and the toggle allows the movable wedge to be lowered.
3. In the meantime, the leaf moves in the downstream direction, transferring the water load from the roller to the gate seat.

Sealing is accomplished by metal-to-metal contact in exactly the same manner as in the simple slide gate. To open this gate the movable wedge is raised first, this move being made positive by the design of the toggle mechanism. As the wedge is raised, the gate moves upstream away from the seat, this time transferring the water load from the seat to the roller trains. The leaf is then raised on purely rolling surfaces. In short, the gate is raised or lowered as a roller gate and sealed as a slide gate.

Model studies.

Description of model.—In order to check the mechanics of the gate under working conditions, a model was constructed to a scale of 1: 28. The prototype gate was to operate on the face of the dam under 176 feet of water. Since an exact duplicate set-up for mechanical or hydraulic tests, even at the 1: 28 scale, would have required a large tank and overflow and would have made it almost impossible to observe the action of the leaf, a housing fitted with windows was used.

Flat-bottom gate.—After testing and checking the mechanics of the leaf, a number of tests were made on the hydraulics of the gate. In order to determine the pressures and forces acting on the gate, a series of piezometers was attached to the leaf and a spring scale was attached to the stem, the rope hoist having been replaced by a stem for the tests.

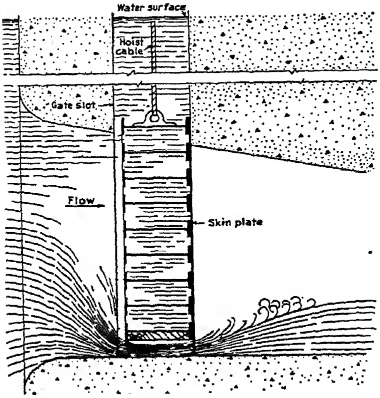


FIGURE 354.—*Submerged gate.*

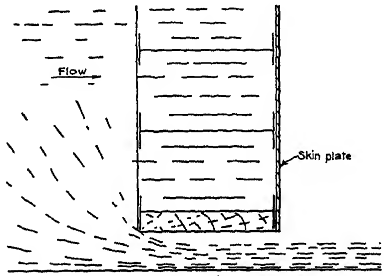


FIGURE 355.—*Flat-bottom gate.*

The gate shown in figure 354 has been raised to such a point that the distance from the bottom of the gate to the seat is slightly less than one-third the thickness of the gate. At such a position, the section under the gate is similar in effect to a short tube. If the head on the gate is relatively low, less than 40 feet, the stream leaving the back edge of the gate is agitated and divergent and its force is quickly dissipated. As there is practically no flow along the upstream face of the gate, the water pressure above and below each cross beam, except the bottom one, is very nearly balanced and is equal to the static pressure head on the respective beams. The pressure above the bottom beam is very nearly the static water pressure at that point, while the pressure below this beam is less than atmospheric (short tube effect). This pressure difference applies a downward force on the leaf of slightly more than the operating water pressure on the bottom beam multiplied by the area of the bottom of the gate.

Considering the same gate but operating under a higher head, the stream will take more the shape of a jet being discharged from a sharp-edged orifice, the short tube effect breaking down as the velocity increases. The stream, being no longer broomy, will continue in a fairly solid, clear jet for some distance after leaving the upstream edge of the leaf. (See fig. 355.) As the discharge pipe fills with water, submerging the jet, there is a change in the shape of the jet and also in the coefficient of discharge. At partial gate openings, with the discharge pipe flowing full, there is a partial vacuum formed back of the leaf. This may be reduced by installing an air vent in the discharge line near the leaf. The pressure below the leaf at free discharge is atmospheric, but it is less than atmospheric if the jet is just submerged or if there is a partial vacuum in the discharge pipe. With the pressure below the bottom beam at atmospheric or less, as before, the top of the beam is practically under full headwater pressure. The downpull in this case is also equal to slightly more than the operating water pressure on the bottom beam multiplied by the area of the bottom of the gate. The maximum downpull on a flat-bottom gate occurs at a gate opening equal to about one-third of the thickness of the leaf.

Sloping-bottom gate.—Downpull can be greatly reduced by changing the shape of the bottom of the gate as shown in figure 356. If this leaf is raised the same amount as in the two previously mentioned gates, the distance X becomes the depth of the controlling discharge area. The remainder of the gate leaf is at various distances from the seat, and therefore a given amount of water will have a changing velocity as it passes under the leaf. At point B, ignoring losses and contractions, the velocity is half that at A. As the velocity varies as the square root of the head, only one-quarter of the static head at B has been converted into velocity head. Therefore, the uplift or pressure at B is equal to three-quarters of the head on the gate. As the leaf is raised, the ratio of openings at B and A changes from an infinite value at complete closure to 2 to 1, as shown, and to nearly 1 to 1 as the gate is completely opened. At a nearly 1 to 1 ratio, the sloping bottom leaf is a little better than the flat-bottom leaf, but as a rule the gross area of the gate opening is large compared to the opening of the far end of the discharge pipe; therefore, the pipe will flow full and form a back pressure before the leaf is completely raised. In this way the back pressure comes into play about the time the major effect of the sloping bottom is lost.

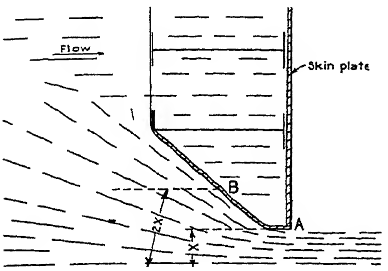


FIGURE 356.—*Sloping-bottom gate.*

The pressure on the bottom of the leaf may be further increased by changing the shape of the bottom of the frame and approach, as shown in figure 357. In all cases it will be noted that the large back area of the controlling surface has been increased. The downpull, due to unbalanced pressures at any gate opening, may be approximated on the basis of varying velocity heads below the gate leaf.

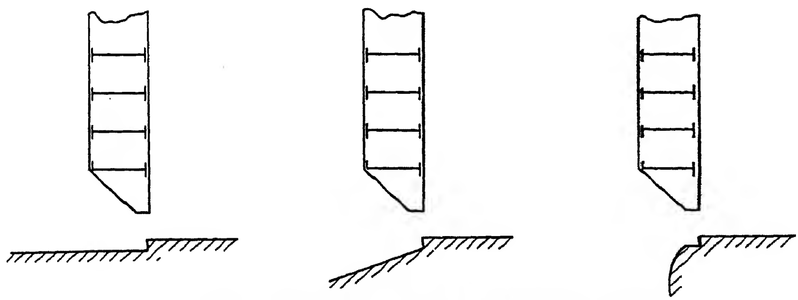


FIGURE 357.—*Various shapes of bottoms and approaches.*

All the above conditions are based on a submerged leaf having the skin plate on the downstream side. If the gate is submerged, but with the skin plate on the upstream side, the unbalanced pressure will be between the top and bottom of the gate. If the gate has a free water surface and has the skin plate on the downstream side, the above conditions for the bottom beam will still hold; but if the skin plate is on the upstream side, only atmospheric pressure acts on the top of the gate, and then the bottom beam should either be vented or shaped so as to prevent a less than atmospheric pressure under the bottom beam.

Test results.—Table 161 contains test data on various-shaped bottom beams: I–beam, T–shaped, and 45° slope types. It will be noticed that the pressure above the bottom beam drops sharply in the model test; actually this pressure on the face of the dam should drop by about the amount the gate is raised

In this study the pump was so small that at one-half gate opening the line pressure had dropped nearly 80 percent. Such a test, to get prototype scale, would require a large constant head tank. Figure 358 shows graphically the test results on the three bottom-beam types mentioned and indicates pressure above and below the beams

TABLE 161.—*Tractor gate model study*

Movement of gate in inches	Type bottom beam: Pressure above bottom beam (pounds per square inch)			Average pressure bottom side of bottom beam [1] (pounds per square inch)			Difference of pressures (pounds per square inch)		
	I	T [2]	45° [3]	I	T [2]	45° [3]	I	T	45° [3]
0	5.90	6.10	6.30	5.90	6.10	6.30	0.00	0.00	0.00
Wedge raised	5.70	5.90	6.00	5.50	5.80	6.00	.20	.10	.00
½	5.20	5.50	5.40	4.70	5.10	5.25	.50	.40	.15
1	4.40	4.80	4.15	3.30	3.80	3.50	1.10	.90	.65
1½	3.40	3.70	3.00	1.40	2.10	2.20	2.00	1.60	.80
2	2.65	2.90	2.20	.25	.70	1.30	2.40	2.20	.90
2½	2.10	2.40	1.90	−.20	.00	1.00	2.30	2.40	.90
3	1.80	2.00	1.65	−.35	−.10	.85	2.15	2.10	.80
4	1.50	1.70	1.35	.00	.40	.80	1.50	1.30	.55
5	1.35	1.50	1.25	.25	.70	.85	1.10	.80	.40
6	1.30	1.40	1.20	.50	.80	.90	.80	.60	.30
7	1.20	1.35	1.15	.65	.95	.95	.55	.40	.20
8	1.15	1.30	1.10	.80	1.05	.95	.35	.25	.15
10	1.10	1.25	--------	1.00	1.20	--------	1.00	.05	--------

[1] In case of 45° slope on bottom beam, the pressure on the bottom side of bottom beam is the average vertical pressure.

[2] In the above runs the gate was raised ¼ inch before the bottom of the leaf was even with the top of the bottom frame seat.

[3] Pressures on the bottom are the results of three piezometers, giving a fair average pressure. The gate was raised ¼ inch before the bottom of the leaf was even with the top of the bottom frame seat.

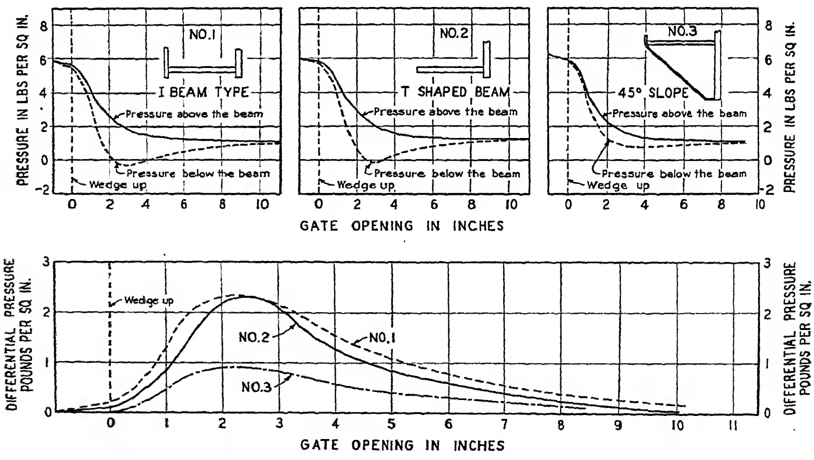


FIGURE 358.—*Pressures above and below beams for the three types tested.*

Figure 358 also offers a comparison of the three types showing differential pressure between the bottom and top of the bottom beam plotted against gate openings.

Tests on tractor gate.

In order to make a comparison of the actual downpull with calculated loads for similar conditions, tests were run on the Norris tractor gate. Four test runs were conducted in the following sequence:

1. Gate lowered under static conditions.
2. Gate raised under static conditions.
3. Gate lowered under full head and full discharge of turbine.
4. Gate raised under static conditions.

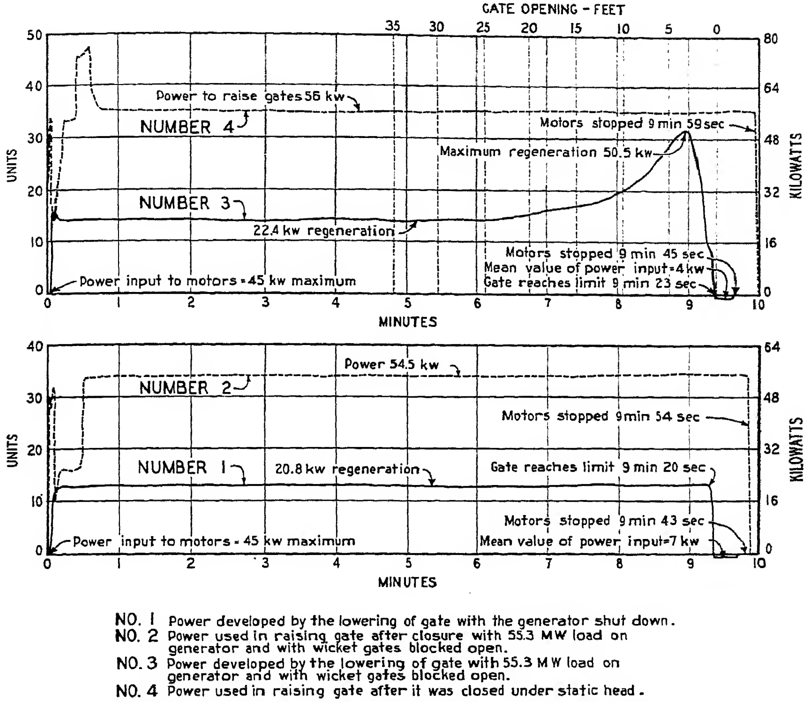


FIGURE 359.—*Recording wattmeter record—Power to lower and open gate.*

During each run a record was kept of the movement of the bottom of the gate with respect to time, and a recording wattmeter record was made of the power developed in the cases where the gate was lowered, or the power used in cases where the gate was raised. Figure 359 shows the wattmeter records for the four runs, while figure 360 is a graphical representation showing closing speed for various gate elevations. Power developed by the run represented by figure 360 is shown by curve No. 3 of figure 359.

Calculations from test data.—Maximum downpull was computed from the test data as follows:

Weight of moving parts in air (estimated), 240,000 pounds.
Weight of moving parts in water 6.8/7.8 x 240,000=209,000 pounds.
Use 210,000-pound weight in water.

From curves Nos. 1 and 2, figure 359.

Thirteen units represent power developed lowering the gate under balanced conditions.

Thirty-five units represent power used in raising the gate under balanced conditions.

Therefore:

210,000—friction=13 units.
210,000+friction=35 units.
2 friction=22 units.
Friction=11 units.
35—11=24 units=210,000 pounds.
1 unit=8,750 pounds.
11 units=96,250 pounds=friction.

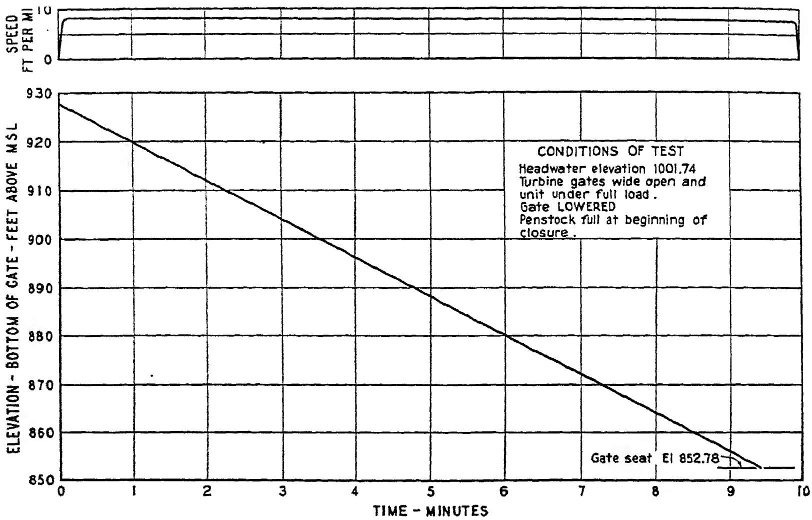


FIGURE 360.—*Closing speed.*

For all calculations the friction is assumed to be proportional to the load. In this computation the friction is 11/24 or 0.458 of the load.

From curve No. 3, figure 359

Fourteen units equal load on hoist before gate reached the opening. Therefore, the increased load on the cable or downpull in any position is equal to: (reading in units at that position—14) ×1.458×8,750. Maximum downpull equals (32—14) ×1.458×8,750=230,000 pounds.

Gate openings are marked on curve No. 3, figure 359. It will be noted that the curve crosses the 14-unit line at 1-foot opening or 1 foot from the gate limit. On the downpull gate opening curve this position is considered zero gate opening. This 1 foot should indicate the distance from the sill to the stop. Actually the sill is only 8 inches above the stop. This difference and possibly a slight difference in the position of the maximum downpull is probably due to the stretch in the cable.

Calculation of theoretical downpull.

To afford a comparison with the test results just outlined, the theoretical downpull on a Norris tractor gate was calculated under these conditions:

Headwater elevation, 1,001.74 feet.
Gate stop elevation, 852.78 feet.
Sill elevation, 853.45 feet.
Tail water elevation, 833.00 feet.

1,001.74—833.00=168.74 feet=head on turbine.
Load on turbine=55,300 kilowatts.
Assume 85 percent efficiency,

$$Q=\frac{55{,}300\times 746}{62.5\times 168.74\times 0.85}=4{,}600 \text{ c. f. s.}$$

Area of wicket gate based on coefficient of discharge of 1.0,

$$A=\frac{Q}{\sqrt{2gh}}=\frac{4{,}600}{8.02\sqrt{168.74}}=44.2 \text{ sq. ft.}$$

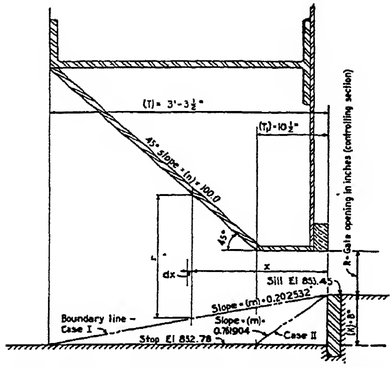


FIGURE 361.—*Dimension of bottom of gate.*

This gate area, and a coefficient of discharge of 0.90 through the tractor gate opening, were used in calculating the pressure drop through the gate.

Downpull is defined as the difference in hoist load between operating the gate under balanced conditions and under emergency or free discharge conditions. This difference is caused by a reduction of water pressure acting upward on the bottom of the gate leaf.

Figure 361 shows the dimensions of the bottom of the gates. Assuming the bottom I-beam of the gate is perforated, then the full static head will be exerted at (1). Let (2) be any section below the gate leaf.

Then writing Bernoulli's theorem from position (1) to (2),

$$H_{(1)}=h_{(2)}+\frac{V_{(2)}^2}{2g}+F_{(2)}.$$

Where: $H_{(1)}$ = unit pressure acting downward, in feet of water.
$h_{(2)}$ = unit pressure acting upward, in feet of water.
$V_{(2)}$ = velocity of water flowing under gate, in feet per second.
$F_{(2)}$ = friction losses, in feet of water.

Friction losses are being ignored in this computation.

Therefore:

$$H_{(1)}-h_{(2)}=\frac{V_{(2)}^2}{2g}$$

or the difference in unit pressures is equal to the velocity head of the water passing below the gate. Then the summation of the velocity head below the gate leaf is equal to the downpull on the gate.

Also, the velocity of the water flowing under the gate is given by:

$$V=\frac{Q}{CA}$$

Considering a unit width of opening of depth, r,

$$V=\frac{Q}{Cr}$$

also

$$V=\sqrt{2gh} \text{ or } h=\frac{V^2}{2g}$$

substituting

$$h=\frac{Q^2}{2gC^2r^2}=\frac{\text{constant}}{r^2}$$

or the head is inversely proportional to the square of the depth of the opening. Since the depth of the opening varies under the gate leaf, the velocity head will vary inversely as r^2.

In order to calculate the downpull which is equal to the summation of the velocity head of the water flowing under the gate, the effective thickness of the gate having a zero upward pressure is computed.

Let T_e = effective thickness of gate.
T = thickness of gate = 39½ inches.
R = depth of gate opening at the controlling section.

Then

$$T_e=\frac{R^2T}{\Sigma r^2}$$

The downpull is then equal to the effective thicknes (T_e) multiplied by the width of the gate opening in inches and the pressure drop through the gate in pounds.

It will be noted that in raising such a gate the effective width of the gate approaches the actual width of the gate, and in the case of a free discharge the downpull will approach that for a flat-bottom gate.

Calculations, case I, based on boundary line (I)

Let m = slope of boundary line.
n = slope of gate bottom.
x = distance from controlling section.

Then

$$T_e=\int_0^{T_1}\frac{R^2dx}{(R+mx)^2}+\int_{T_1}^{T}\frac{R^2dx}{[R-T_1+(m+n)x]^2}$$

$$=\frac{-R^2}{m(R+mT_1)}+\frac{R^2}{mR}-\frac{R^2}{(m+n)R-T_1+(m+n)T}+\frac{R^2}{(m+n)(R+mT_1)}$$

Substituting values for m, n, T, and T_1 and combining terms,

$$T_e=R^2\left[\frac{-4.105912}{R+2.126582}+\frac{4.937491}{R}-\frac{0.831579}{R+37.000}\right]$$

Example: Let R = 24 inches.
T_e = 576 (−0.157155 + 0.205729 − 0.013632).
T_e = 20.13 inches or effective width of gate.
Downpull = $P\times0.433\times T_e\times W$
= 127 × 0.433 × 20.13 × 198
= 219,180 lb.

Where P = pressure drop hrough gate in feet of water.
W = width of gate opening in inches.

The effect of regain of velocity head in the penstock was neglected.

Calculations, case II, based on boundary line (II)

$$T_e=\int_0^{T_1}\frac{R^2dx}{(A+mx)^2}+\int_{T_1}^{T}\frac{R^2dx}{[R+K+(x-T_1)n]^2}$$

$$=\frac{-R^2}{m(R+mT_1)}+\frac{R^2}{mR}-\frac{R^2}{n(R+K-nT_1+nT)}+\frac{R^2}{n(R+K)}$$

Substituting and combining:

$$T_e=R^2\left[\frac{-0.312502}{R+8.000}+\frac{1.312502}{R}-\frac{1}{R+37.000}\right]$$

Example:

Let R = 24 inches.
T_e = 576 (−0.009766 + 0.054688 − 0.016393).
T_e = 576 × 0.028529.
= 16.43 inches.

Downpull = P × 0.433 × T_e × W
= 127 × 0.433 × 16.43 × 198
= 178,893 pounds

Discussion of results.

The test results as well as the calculated values for cases I and II are plotted on the curves of figure 362. It will be noted from the downpull gate opening curves that the downpull as shown by the test curve reaches a higher maximum than either of the calculated curves, and that there is a separation of the curves from the maximum position until a gate opening of about 13 feet is reached. Reasons for the difference may be:

1. Neither of the boundary lines used is correct but is merely an approximation. The boundary lines will not remain constant for all gate openings.
2. The coefficient of discharge was assumed to be 0.90 and constant for all gate openings. The coefficient is probably a variable, especially before and after the penstock has filled.
3. Regain of velocity head was neglected; this is probably the greatest source of error.

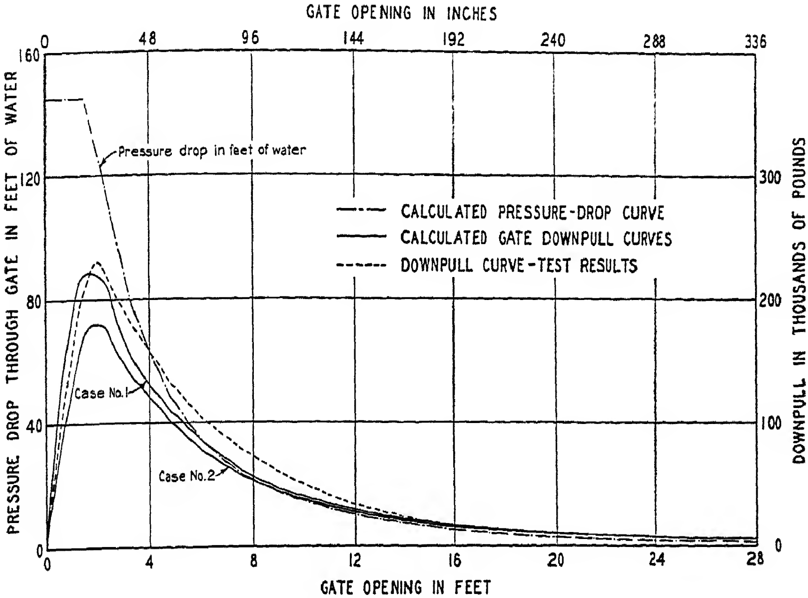


FIGURE 362.—*Test results and calculated values for cases I and II.*

4. All calculations are based on a stabilized flow for each position, but under actual conditions the effect of the gate being continually lowered will make some difference in downpull.
5. Stretch in the cables may shift the position of gate openings slightly.
6. Roller friction has been neglected.

In spite of the above listed possible sources of error, it is believed that downpull can be calculated in this manner consistently within 15 percent of the actual value.

Curve No. 4 of figure 359 (raising gate balanced condition) apparently shows that the penstock was not completely full. Note the character of the curve until a balanced condition was reached.

Appendix E

CEMENT AND AGGREGATE STUDIES

CEMENT STUDIES

The ordinary cement price structure was not satisfactory to the Authority for the large quantities of cement needed for all projects due to the special specification requirements. The Authority, under pressure of assembling an organization and starting construction, sought the advice of Mr. John Treanor, president of Riverside Cement Co., Los Angeles, Calif., and others who were active at that time in the general cement policy and its relation to the Government. There were retained to act as a committee to cooperate with the Authority, Mr. Morris Leeds of Philadelphia, Pa., and Dr. Frank A. Fetters of Princeton University. This committee's activities were confined to the early part of 1934, and after conferences with a committee of cement manufacturers, submitted a report on the cost of manufacture of cement in a modern mill. Their conclusions were based on the average cost of the lowest 10 of the 15 mills in the Southeast, determined from certain combined costs assembled by Price, Waterhouse & Co.

The original cement studies and specifications were made by the United States Bureau of Reclamation at Denver. The first advertisement for bids, calling for 25 percent of the total cement to start construction of Norris Dam, specified as alternates standard portland cement, type A low-heat cement, and type B modified portland cement. Analysis of bids received March 19, 1934, revealed no difference in price among the three types of cement except for one bid in which there was a difference of 5 cents per barrel for type A over other cement.

It was decided by the Authority and concurred in by the Bureau of Reclamation to use modified portland type B cement for the original contract, which cost $1.95 per barrel f. o. b. cars Coal Creek. This type of cement was also endorsed by the Portland Cement Association. The reasons for the choice of this type of cement were:

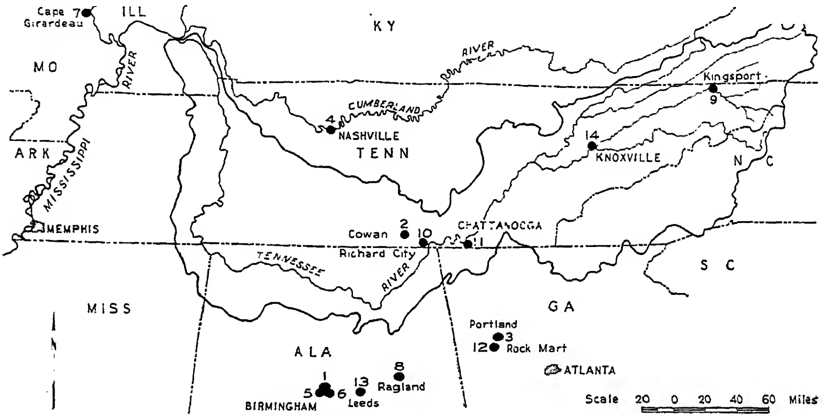
1. The property of low-heat generation was not a factor in determining the specifications for type B cement as early studies showed the difference in the estimated maximum temperatures for low-heat and normal portland types to be only 2° F.
2. The specifications for type B cement were intended to provide an all-purpose cement having strength development characteristics suitable for winter and summer construction of all types, maximum resistance to weathering and dissolution, favorable volume change characteristics, and properties resulting in favorable concrete workability.

The unusually large quantities involved made it desirable to make studies to secure the most economical source of cement. A survey was made to determine the facilities available in the Tennessee Valley area for supplying the cement. Consideration was given to the leasing of certain cement manufacturing plants for a period of years and manufacturing by force account methods the cement required. Some consideration was given to the purchase of a small cement plant near the center of the Authority's operations. A study was also made of nitrate plant No. 2 at Muscle Shoals with the view of making modifications in order that the rotary kilns and mills might be used for the manufacture of cement. However, the total cost, including transportation of rock from the quarry, manufacture of cement, and the transportation of the cement to the project, would have been higher than the cost of purchasing cement.

The most attractive scheme was the construction of an entirely new plant. A location had been found about 5 miles below Sheffield where there was a deposit of limestone of exceptionally high quality with an overburden of materials which could be utilized in the manufacture of cement. The deposit, in a vertical

cliff, could be quarried very cheaply; and between the cliff and the river was a desirable location for a plant. A complete plant for this site was designed which would have been equal or superior to any plant in the United States. It would have been relatively simple due to one type of product and one outlet. Complete specifications were prepared with a view to constructing it in the shortest possible time.

Representatives of the F. L. Smidth & Co. and the Allis-Chalmers Manufacturing Co. cooperated in the design; and the specifications were reviewed by them to make certain that if and when bids were asked for construction of the plant, including all equipment, there would be at least two companies which would be able to bid on such a complete plant. A survey at that time indicated that there were no other companies which were equipped to undertake the construction of such a cement plant in its entirety. The specifications as written called almost entirely for standard equipment. The estimated cost of the plant was determined from preliminary estimates of the cost of machinery furnished by cement machinery manufacturers.



ANNUAL CLINKER CAPACITY IN THOUSAND BARRELS

1	ALPHA PORTLAND CEMENT CO.	1475	8	NATIONAL CEMENT CO.	1320 *
2	CUMBERLAND PORTLAND CEMENT CO.	840	9	PENNA-DIXIE CEMENT CORP.	1380
3	GEORGIA CEMENT & PRODUCTS CO.	435 *	10	" " " "	1985
4	HERMITAGE PORTLAND CEMENT CO.	865	11	SIGNAL MT. PORTLAND CEMENT CO.	1495
5	LEHIGH PORTLAND CEMENT CO.	1904	12	SO. STATES PORTLAND CEMENT CO.	590 *
6	LONE STAR CEMENT CO.	1690	13	UNIVERSAL ATLAS CEMENT CO.	2015
7	MARQUETTE CEMENT MFG. CO.	1650	14	VOLUNTEER PORTLAND CEMENT CO.	980

FIGURE 363.—*Cement plants serving the Tennessee Valley.*

An estimate of the cost of manufacturing cement in this proposed plant was made. It included the quarry operation cost (based on the Authority's operations at the Norris Dam quarry, modified to meet local conditions), power and fuel consumption (based upon guaranteed production from cement-machinery manufacturers), the necessary personnel, and interest and amortization (based on fully amortizing the plant in 5 years with no salvage value at the end of this period).

This estimated cost of manufacturing cement was checked with the cost of manufacturing cement at the 10 mills in the Tennessee Valley which had been compiled and certified by Price, Waterhouse & Co.

A committee of nearby cement manufacturers supplied the Authority with cost data in order to justify their established prices. The Authority in return submitted to them an estimated cost of the proposed new plant together with the estimated cost of operation. These estimates were thoroughly reviewed

over a period of 30 days by a technical committee appointed by the cement committee, and comparisons were made with their own actual cost figures. It was agreed that the proposed plant was feasible. The Authority's estimates and studies of the necessary transportation equipment and its operating cost were also reviewed by traffic men in the cement industry.

While the above studies indicated the Authority could manufacture cement more cheaply than it could be bought, it was considered unwise to build a new plant in an already overcrowded industry. The total clinker capacity of the 14 mills shown in figure 363 is 18,634,000 barrels, yet the total cement production for the States of Alabama, Mississippi, and Tennessee for 1933, excluding Tennessee Valley Authority construction, was only 2,567,810 barrels.

Not desiring to add to the overcrowded industry by building still another plant, the Authority expressed a willingness to pay a price for cement somewhat higher than the indicated costs of manufacture in the proposed new plant yet lower than the cement manufacturers considered fair. The cement manufacturers were advised of the Authority's decision in the following telegram dated September 18, 1934:

"Authority intends not to build a cement plant if cement manufacturers quote a base price not to exceed one dollar and thirty cents per barrel at mill plus freight to destination. Where only one mill enjoys the lowest freight rate will consider above base price plus freight from second lowest freight point. It is to be understood that this is maximum that the Authority will consider and any quotations should be at or below figure mentioned. It is our intent to build a cement plant if average price omitting Holston and French Broad Dams is higher than this rather than name a lower price with intent of negotiating small differences. We are stating the outside maximum. If any differences of understanding remain we suggest they be cleared up by conference before bids are submitted. It is also understood that any change in freight rates will be adjusted. * * *"

The experience in the use of type B modified portland cement during the summer of 1934 at both Norris and Wheeler Dams was uniformly satisfactory. Therefore specifications for the balance of Norris and Wheeler requirements and the full requirements of all other Tennessee Valley Authority projects covered only type B modified portland cement with small quantities of standard portland cement. It was anticipated that type B cement would be used on all dams and construction work except where it was necessary for architectural reasons to have a lighter-colored cement, in which case standard portland cement was to be used.

Bids were received October 15, 1934, at the price the Authority had indicated its willingness to pay. An allocation for the requirements of cement was made with the view of distributing the cement to as many manufacturers as possible, taking care to see that business was uniformly distributed to the manufacturers throughout the life of the contracts and also taking care that no manufacturer was unduly penalized due to freight rates.

Contracts were placed with 10 companies covering 11 mills for the entire requirements of TVA up to January 1, 1941. Due to the uncertainty in 1934 as to the probable price trend, it was deemed desirable that certain provisions be made in the contract to compensate either the Authority or the contractor for any exceptional price fluctuations in fuel, power, labor, and materials (iron ore and gypsum). This was accomplished by setting up a definite formula based on the then current prices of these major items which went into the manufacture of cement. The contract also provided that changes in transportation cost be added to or deducted from the delivered cost.

The price per barrel in bulk, f. o. b. Coal Creek, was $1.7384, subject to 10 cents per barrel discount for prompt payment. The total amount of business allocated to each contractor for the Norris project was:

	Barrels
Volunteer Portland Cement Co.	587,800
Pennsylvania-Dixie Portland Cement Co.	396,600
Signal Mountain Cement Co.	249,300

The estimated average mill net of all contractors on all projects was approximately $1.30 per barrel for the contracts placed in October 1934. The considerations which influenced the distribution of cement purchased by the Authority were:

1. It was important that purchases be made from plants conveniently located for shipping delivery and from plants financially and physically equipped so that the progress of concrete construction should not be jeopardized by interruptions in delivery.

2. Within reasonable limits, those mills were favored which had the lowest freight charges and the largest mill net prices, as in that way the cement industry as a whole would profit most from the orders being placed.

3. There was a distribution of business among reasonably well-situated plants so that the cement industry would have a fairly smooth flow of business in the several mills rather than a series of violent fluctuations in production.

4. Other conditions being equal, the distribution of business in a period of low production was somewhat in proportion to the capacities of the several plants.

5. In the original order of the Authority, certain plants within shipping distance were given no business. It therefore seemed appropriate to assign to these plants somewhat more than their proportion of the second order.

6. The program for four dams seemed immediately practicable, whereas certain orders might be postponed for several years. The distribution was such that it would meet the above conditions for the dams to be built immediately regardless of the more remote parts of the program.

7. While the above factors were in general controlling, various others were weighed in special cases such as the wisdom of buying from nearby mills so that trucking could be resorted to in case of marked advances in freight rates, and also the possibility that general advances in freight rates might affect the cost of delivered cement from the more distant plants. In general, however, these speculative factors were given little weight.

The contracts operated in a very satisfactory manner. The price adjustment clause was not effective except twice, and then affecting a relatively small amount of cement. The comments from officials of the cement companies were uniformly complimentary.

TYPE B—SLAG CEMENT INVESTIGATIONS

Investigations were conducted to determine the merits of a mixture of slag cement and portland cement in mass concrete. Durability of concrete containing the mixture was one of the doubtful qualities, and for direct comparison under field conditions one construction block in the dam was placed using a 25 percent replacement of modified portland type B cement with a blast furnace slag cement. In the absence of definite data on the most desirable proportioning of the two cements, the 25 percent replacement noted was arbitrarily chosen.

Use of the mixture of cements in concrete of one entire block afforded an excellent large-scale comparison of such properties as durability, workability, temperature rise, and cracking of such concrete with the concrete containing type B cement only, used in all other parts of the dam. Block 43 above elevation 870 was constructed entirely of concrete using the mixture of cements. Block 40, having a very similar concrete schedule, was used for comparisons such as temperature rise and cracking. Test samples of concrete taken from each of the two blocks furnished comparative strength data, since, in general, identical bins of type B cement went into the concrete of both blocks.

In proportioning the concrete mix using slag cement, a replacement of type B cement was made on a basis of one barrel (320 pounds) of slag cement being equivalent to one barrel (376 pounds) of type B. Thus one barrel of the mixture weighed 362 pounds. Water-cement ratio of concrete using the mixture of cements was varied to give workability equal to equivalent concrete containing only type B cement. Aggregate gradings were identical for equivalent mixes. The mixes for a 1-cubic-yard batch developed for use in block 43 together with mixes containing only type B cement are shown in table 162.

The slag cement was an interground blend of blast-furnace slag and hydrated lime, manufactured by the Southern Cement Co. of Birmingham, Ala., and sold under the trade name of Magnolia cement. Type B cement was the standard product furnished under Tennessee Valley Authority specifications. Average oxide analyses of both cements are shown in table 163. The analyses shown include the added lime. Fineness of the slag cement was greater than 2,600 square centimeters per gram by Wagner turbidimeter.

TABLE 162.—*Mix data for a 1-cubic-yard batch*

		Cement	Type B cement	Slag cement	Water	W/C ratio (by weight)	Aggregate	Maximum size aggregate
		Barrels	*Pounds*	*Pounds*	*Pounds*		*Pounds*	*Inches*
Mass	A	0.90	338		227	0.67	3,772	6
Concrete	B	.90	254	72	221	.68	3,790	6
United States face	A	1.10	414		240	.58	3,660	6
Concrete	B	1.10	310	88	235	.59	3,684	6
Spillway	A	1.20	451		253	.56	3,596	6
Face concrete	B	1.20	338	96	248	.57	3,615	6
Reinforced	A	1.33	500		275	.55	3,489	3
Concrete	B	1.33	375	106	270	.56	3,509	3
Reinforced	A	1.50	564		310	.55	3,332	1½
Concrete	B	1.50	423	120	304	.56	3,355	1½

A denotes standard concrete mix using type B cement.
B denotes concrete mix for block 43 using mixture of type B and slag cement.

TABLE 163.—*Chemical analyses of cements*

Item	Slag cement Block 43	Type B cement	
		Block 43	Block 40
Ignition loss	2.52	0.85	0.94
Insoluble		.21	.22
Sulphur (S)	1.18		
Sulphur trioxide (SO_3)	.02	1.62	1.69
Silicon dioxide (SiO_2)	32.11	21.54	21.54
Ferric oxide (Fe_2O_3)	.55	4.80	4.84
Aluminum oxide (Al_2O_3)	9.83	5.53	5.58
Calcium oxide (CaO)	51.06	63.64	63.66
Magnesium oxide (MgO)	1.84	1.86	1.82

The batching, mixing, and placing operations were identical with procedure developed for all concreting operations in the dam. Results of mass concrete strength tests on 6- by 12-inch cylinders subjected to standard curing are shown in table 164.

TABLE 164.—*Mass concrete strengths blocks 40 and 43*

Age	Block 40 (type B)		Block 43 (slag mix)	
	Number	Average strength	Number	Average strength
7 day	18	2,870	18	2,370
14 day	16	3,790	16	3,440
28 day	35	4,540	36	4,500
90 day	34	5,810	35	6,120
6 months	36	6,371	36	6,391
1 year	36	6,846	36	6,997
2 years	19	7,268	19	7,563

Approximately 18 cylinders each are to be tested at ages of 5, 10, and 20 years.
All specimens made of full mix wet screened to 1½-inch maximum size. Rate of loading, 17 pounds per square inch per second.

Temperature rise in the two blocks at elevation 910 was investigated by means of resistance thermometers similarly embedded in the concrete and the data are listed in the following table. Conditions in adjacent blocks were such that more heat was removed from block 40 than from 43; consequently temperature rise did not exactly represent differences in heat liberation of the cements, particularly after ages of 28 days. Opposing this, however, was the effect of a 30-inch vent pipe in block 43 approximately 10 feet above the layer of thermometers for half of the width of the block which removed a small but

unknown amount of heat. Since conditions at the downstream faces were not comparable, only the upstream 87 feet of each block enters into the temperature study.

TABLE 165.—*Concrete temperatures blocks 40 and 43*

[Degrees Fahrenheit]

Age (days)	Block 40 (type B) average temperature (upstream 87 feet only)				Block 43 (slag mix) average temperature (upstream 87 feet only)			
	East half	West half	Entire block	Near center of block	East half	West half	Entire block	Near center of block
0	82.0	82.0	82.0	82.0	81.0	81.0	81.0	81.0
9	112.7	113.7	113.2	115.9	111.5	113.7	112.6	115.6
15	114.5	115.0	114.8	118.6	113.1	114.3	113.7	117.2
28	114.4	114.7	114.5	121.5	112.2	112.6	112.4	119.5
60	109.6	110.3	110.0	122.0	107.5	112.7	110.1	120.0

It appears that temperature rise in the mass concrete was slightly less for concrete containing the mixture of cements than it was for straight type B cement. Heat of hydration, measured by vane calorimeters in the Norris Dam laboratory for type B cement, slag cement, and a mixture of the two would indicate substantially the same results.

There was little or no difference in the cracking which developed in the two blocks. Both blocks cracked vertically between contraction joints on both upstream and downstream faces, and horizontally at several elevations at day's work joints. Several cracks developed in the block 43 spillway training wall at approximately right angles to the face of the dam. Equal or slightly less cracking occurred in the block 37 training wall.

Present results are insufficient to draw definite conclusions. Durability comparisons will not be possible for many years, and accurate determination of comparative costs will not be possible until all of the factors determining quality of concrete are known.

SPECIAL LABORATORY STUDIES

In addition to routine testing, a number of special studies and tests were made in the concrete laboratory. The more important of these are discussed in this appendix.

Moisture volume change.

In investigating the effect of the minus 100 mesh dolomite on drying shrinkage, the mortar mixes selected for the tests were based upon the sand grading recommended by the United States Bureau of Reclamation Laboratory at Denver, Colo. The amount of minus 100 mesh material was varied from 0 to 22 percent, and the relative proportions of the remaining sizes, from plus 100 mesh to minus 4 mesh, were held constant at the recommended values. To minimize the effect of all factors except the variation in sand grading, the cement content was made rather low, 20 percent of the weight of the aggregate, and the water-cement ratio was the lowest which gave a readily workable mix. The specimens were 3- by 3- by 14-inch bars with strain-gage plugs 10 inches apart. All specimens were stored in the 70° fog room from the time of casting until initial measurements were taken at the age of 24 hours.

Although test results indicated a rough direct relationship between drying shrinkage and fineness of the aggregate, the effect was not pronounced, even for the extreme variations in the mortar mixes. Therefore, it is believed the effect of moderate variations in the amount of minus 100 mesh aggregate in a concrete mix would be negligible.

Strength and workability of laboratory mixes.

To arrive at some general impressions as to appearance, workability, strength, and general qualities of concrete containing Norris dolomite and modified type B portland cement, laboratory studies were made, covering roughly the mixes to be used in the dam. Twelve batches of concrete were mixed in the labora-

tory mixer using different size aggregate, varying amounts of cement, and different water-cement ratios. Test cylinders from the resulting concrete molded were given standard cures and tested for compressive strength.

The effect of water-cement ratio on concrete containing dolomite aggregate and type B cement was investigated over a range between 0.40 and 0.85 by weight. Figure 364 shows the water-cement ratio strength relation for an age of 28 days. Cement content was varied from 1.00 barrel per cubic yard for water-cement ratio of 0.85 to 1.60 barrels for water-cement ratio of 0.40. The effect of cement content over the same range with constant water-cement ratio (0.55 by weight), indicated that cement content has only a small effect on strength.

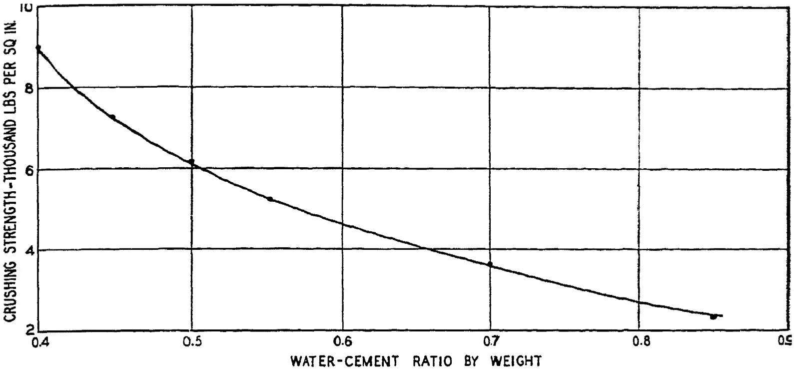


FIGURE 364.—*Strength of laboratory-mixed concrete at 28 days.*

Permeability.

Permeability measurements on Norris Dam test concrete were made by the United States Bureau of Reclamation, and test data are given in the Bureau's Technical Memorandum No. 368.

As a rough field check on permeability of concrete, three pairs of 8- by 16-inch test cylinders were prepared. A section of open 2-inch pipe, cast axially in each cylinder, extended to within 2 inches of one end and protruded through the other end for connection to water supply. Damp sand, compacted in the pipe before the mold was filled with concrete, was removed after the concrete had hardened; thus leaving free passage of water to within 2 inches of one end of the cylinder. The test specimens were cured for 60 days in the fog room and then connected to a reasonably constant water pressure of 180 pounds per square inch. In test specimens left under pressure for several months no leakage through the concrete was found. The test samples contained mass, face, and 1½-inch reinforced concrete, each wet-screened to ¾-inch maximum size. As no leakage developed, no comparison of the three mixes could be made.

Type B—Slag cement, laboratory mixes.

Strength comparisons of laboratory mixed concrete were made using all type B cement, a mixture of type B and slag cements, and all slag cement with Norris dolomite aggregate prior to the use of the slag-type B mixture in the concrete for block 43. The strengths shown in table 166 indicate that the laboratory-mixed concrete has appreciably lower strengths than the field-mixed concrete. This was generally true with laboratory and field-mixed concrete. Comparisons were made on a basis of equal water-cement ratios by weight and variable workability with equal "loose volume" of cement contained in all mixes. The samples were molded in 6- by 12-inch cylinders using 1½-inch maximum size aggregate. Rate of loading was 17 pounds per square inch per second.

TABLE 166.—*Compressive strength laboratory-mixed concrete*

Age	Type B portland		All slag cement		Mixture type B and slag	
	Number	Strength	Number	Strength	Number	Strength
7 days	30	2, 016	30	727	30	1, 599
14 days	30	2, 634	30	915	30	2, 331
28 days	30	3, 470	30	1, 105	30	3, 524
90 days	30	4, 785	30	1, 345	30	5, 044
6 months	30	5, 199	30	1, 514	28	5, 556
Average slump	3¼ inches		⅜+ inch		1⅞+ inches	

Experimental heat of hydration apparatus.

In connection with studies on temperature rise in the dam, heat of hydration experiments on several cements were made, and apparatus similar to Carlson's vane calorimeter was developed. Satisfactory results were obtained, and it is believed that further development may provide a simple, easily constructed apparatus for measuring rate of heat liberation of cement paste up to ages of several months.

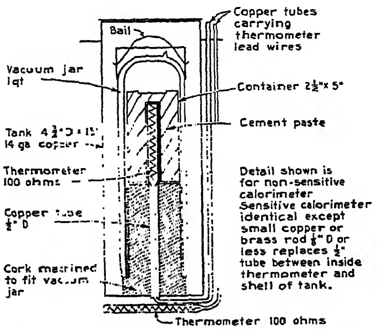


FIGURE 365.—*Modification of Carlson vane calorimeter.*

Operation of the calorimeter shown in figure 365 depends on heat transfer through the copper heat conductor being proportional to temperature differences between ends of the conductor. The two resistance thermometers provide a means for measuring the two temperatures. As thermometer resistances are proportional to temperatures, a change in resistance ratio of the two thermometers was used to represent ratio of heat transfer. Heat stored in the calorimeter was computed from heat capacities of the specimens and the inside of the calorimeter, and corrected measured temperatures. In operation the apparatus was immersed to the flange near its top in a constant temperature bath.

Approximately 500 grams of cement paste to be tested was sealed in the container. The specimen was placed in the calorimeter and the vacuum jar placed in position. Beginning one-half hour later, after temperatures became constant, resistance ratio readings by a special Wheatstone bridge were made at half-hour intervals until the specimen had passed its maximum temperature. Readings were then made as often as needed to furnish a smooth time-ratio curve. After 48 hours in this (nonsensitive) calorimeter the specimen was transferred to a second (sensitive) calorimeter on which readings were continued. After correction for stored heat, areas under the time-ratio curves multiplied by calibration constants of the calorimeters gave quantity of heat liberated by the cement paste while in the calorimeters. "Immediate heat" determined in a simple calorimeter completed the data necessary for determining heat of hydration.

The constant temperature bath was found essential for accurate results. For the sensitive calorimeter, the temperature of bath seldom varied more than 0.01 degree centigrade; variations were somewhat larger for the nonsensitive calorimeter due to more rapid heat transfer from the test specimen.

Effect of cylinder size on strength.

As a matter of general information, tests were made to compare strengths of field-mixed concrete cast in 8- by 16-inch and 6- by 12-inch test cylinders. A total of 100 specimens of each size made from samples of field-mixed mass concrete wet screened to 1½-inch maximum size aggregate was tested at 28 days for compressive strength. The average strength of smaller cylinders was 4,481 pounds per square inch and for the larger was 4,491 pounds.

Calcium chloride in concrete.

Although no accelerator or admixture was used in any concrete for the permanent structures, it appeared to be desirable at times to use a rapidly hardening concrete in certain temporary structures. With this in mind, and as a matter of general information, some laboratory tests were conducted to show the effect of various amounts of calcium chloride ($CaCl_2$) on early strengths of concrete made from type B cement. The mix was 1.70 barrels of cement per cubic yard, with a water-cement ratio of 0.50 by weight, and a maximum size aggregate of 1½ inches was used. The strengths in pounds per square inch are averages of three specimens in the following table summarizing these tests:

$CaCl_2$ percent by weight of cement	Compressive strength		$CaCl_2$ percent by weight of cement	Compressive strength	
	1 day	2 days		1 day	2 days
0	990	1,660	4	2,080	2,910
1	1,440	2,360	6	1,760	2,680
2	1,490	2,460	8	1,560	2,410
3	1,890	2,420			

Mortar workability.

Effect of physical properties of materials and methods of manufacture of sand on workability of comparable mortars is shown on figure 366. Four sands of identical grading were used for the comparison.

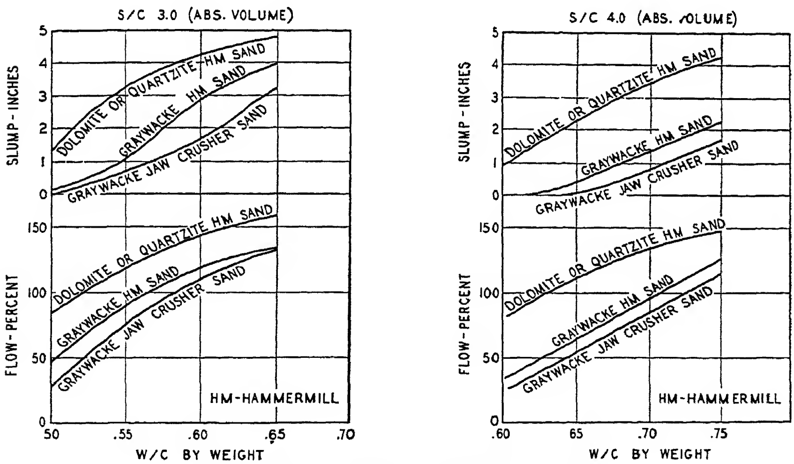


FIGURE 366.—*Mortar workability of sand.*

TESTS ON ROCK FLOUR-CEMENT MIXTURE FOR GROUTING

To determine the suitability of the rock-flour-cement mixture for grouting, tests were made in an attempt to learn the characteristics of the resulting product. These tests indicated that economies could be effected by using a rock-flour-cement grout containing calcium chloride.

The first test was in the form of a trial run of the material through a pipe about 200 feet long, laid with the usual number of fittings and irregularities,

and then returning to discharge into the mixer the grout consisted of equal parts of cement and rock flour and had a water-solid ratio of 1.0. The mixture was circulated from the pump very slowly to allow opportunity for depositing and building up in the pipe line. Every 2 hours, a fresh batch replaced the one in circulation to avoid the loss of chemical properties occurring when cement remains in an agitated solution. Samples of the grout were obtained from the mixer to be observed for rate of setting, hardness, and for a general comparison with cement grout. The test was continued for 48 hours, at the end of which time the pipes and pump were dismantled and found to be bright and clean, altogether different in appearance from what would have been expected if only cement had been used. With cement grout pumped at the same rate, the pipes would have plugged with the material soon after the test was started. The samples that had been taken from the mixer hardened more slowly than regular cement grout.

These preliminary tests indicated that the characteristics of rock-flour-cement grout differed from those of cement grout to a degree that would make it desirable to revise the scheme of grouting if its use was adopted, and established the necessity for laboratory tests supplemented by additional field investigations.

Laboratory tests.

The data contained in table 167, were the results of tests performed in the laboratory. The Bouyoucos hydrometer analysis indicated that rock flour was finer than cement and the moisture-weight determination was obtained for use in proportioning mixes. The time-of-set data were perhaps the most interesting and pertinent as regards the determination of the suitability of the material for grouting. It was definitely proven that the addition of rock flour had a retarding effect upon the time of set of the mixture.

TABLE 167.—*Results of tests performed on rock flour.*

Rock flour—Moisture			Bouyoucos hydrometer analysis—Rock flour	
Percent bone dry basis	Unit weight (pounds per cubic foot) Loose	Unit weight (pounds per cubic foot) Rodded	Particle size, microns	Percent smaller
0	83	93	1	3
5	59	66	5	10
10	60	73	10	17
15	70	81	15	24
20	86	95	20	32
25	122	125	25	39
30	124	124	30	48
35	119	119	35	55
40	114	114	40	63
			97.3% passed No. 200 sieve	

SETTING TIME

Test No.	Mix by volume Cement	Mix by volume Rock flour	Mix by volume Water	Initial set, hours: minutes	Final set, hours: minutes
1	1	-------	[1]	2:10	5:00
2	1	-------	[1] [2]	0:40	2:00
3	1	-------	1½	36:00	[3]
4	1	-------	[2] 1½	24:00	[3]
5	1	1	[1]	3:15	6:50
6	1	1	[1] [2]	2:45	5:00
7	1	1	1½	30:00	[3]
8	1	1	[2] 1½	28:00	[3]
9	2	1	[1]	2:50	6:35
10	2	1	[1] [2]	1:45	3:25
11	2	1	1½	11:00	22:00
12	2	1	[2] 1½	4:05	10:30
13	1	2	[1]	4:30	6:45
14	1	2	[1] [2]	2:15	4:20
15	1	2	1½	16:00	36:00
16	1	2	[2] 1½	10:00	18:00

[1] Normal consistency (water 23 percent by weight of cement, or cement and rock flour).
[2] $CaCl_2$, 3 percent by weight of cement, was added to the water.
[3] Did not attain final set in 6½ days.

Addition of calcium chloride.

Calcium chloride had a tendency to correct the undesirable qualities of the rock flour mixture. The addition of 3 percent of this chemical by weight of the cement accelerated the set to a degree that counteracted the retardation resulting from the use of rock flour. The resulting product possessed characteristics of pumping and handling similar to those of the regular portland cement grout. Compressive tests on specimens removed from the mixer in the field (proportions of 1.0 cement, 1.0 rock flour, and 1.0 water) failed at 600 pounds per square inch at an age of 14 days. These tests also indicate that the product was sufficiently sound to resist erosion. In appearance the product was little different from the regular grout, being dense and apparently impermeable though not quite so hard. Specimens cored from grout-filled seams after the rim grouting was under way failed in compression at 2,000 pounds per square inch at an approximate age of 45 days.

Trial run.

To learn more of the properties of the rock flour grout, a test was conducted in which an effort was made to simulate the conditions surrounding the actual grouting of a seam. Four concrete slabs ground to a flat uniform surface on one side were matched in pairs. Metal shims held the slabs apart 1/16-inch and the grout entered this space through a 1-inch pipe passing through the center of the upper slab. A barrel in which the level of the grout was held constant by an overflow pipe, and into which the grout pumps discharged, served to maintain a constant static head on the slabs. The pressure at the point where the grout entered the 1/16-inch space was observed constantly during the test by means of a manometer. The tests on the different grouting mixtures were run separately, though simultaneously, so that differences in temperature might not affect the comparison. Throughout the duration of the test, and without interrupting the continuity of flow, old grout was replaced by fresh grout at frequent intervals. A record of the temperatures revealed that consistently higher temperatures prevailed in the regular cement grout. The results obtained from a number of tests are summarized below:

Mix by volume			Time required to grout space between slabs, hours	Ratio of grouting time
Cement	Rock flour	Water		
2		2½	19.25	1.00
1	1	2½	64.30	3.34
1	1	[1] 2½	30.30	1.57
2		[1] 2½	9.50	.49

[1] To these mixes were added 3 percent $CaCl_2$ (by weight of cement).

PRELIMINARY FINE AGGREGATE INVESTIGATION

Prior to the selection of the crushing machinery for the production of fine aggregates for Norris Dam, samples of quarry rock were sent to several crusher manufacturers for testing. These rock samples were reduced to pass a standard No. 4 mesh screen in various types of crushers, including roll crushers, rod mills, modified gyratory crushers, and hammer mills. The resulting products were screened and the gradation curves plotted, and samples of the classified sand for each machine were compared as to the shape of the individual particles.

All of the gradation curves followed the same general trend. There was a surplus of the coarse sizes (plus 8 and plus 14 mesh) and a deficiency of the finer sizes (plus 28, plus 48, and plus 100 mesh) when compared to the desired gradation of nearly equal amounts of each size. From this viewpoint, therefore, there was little or no choice as to which machine should be used. It was clearly indicated that improvement in gradation could be effected only by selective screening and recirculation of undesirable oversizes.

A marked difference was apparent in the grain shapes of the products of the various machines. The hammer mill product was cubical, the products of the

gyratory crushers were wedge shape, pyramidal, and flat, while the product of the rolls was composed of splinters. As a cubical shape is the most desirable of the shapes discussed, the hammer mill product was superior. The grain shape of the material from one of the gyratory crushers was quite good; although not cubical, the pyramidal grains were not elongated to a marked degree.

It was claimed that the particle shapes from the cone crusher would be greatly improved under an operating condition where the discharge opening of the crusher was larger than the maximum size of grain in the product desired and the oversize screened out and recirculated for recrushing. The crusher must be choke-fed; that is, the annular feed opening must be filled to capacity. A considerable amount of the reduction would result from attrition grinding or crushing between particles as against crushing between the metal surfaces of the concaves, the latter crushing condition producing flats and undesirable shapes. This claim seemed supported in a plant where a cone crusher was being operated under these conditions. Consequently, the cone crusher was strongly considered for sand production, due largely to its freedom from repairs when compared to the attention required by a hammer mill.

To investigate further the use of this type equipment, a 3-foot Symons short-head cone crusher was rented and installed at the quarry. Tests on it were carried out with specific information being obtained on:

1. Gradation of the sand, or that portion of it passing a No. 4 mesh screen, with special attention to the part passing a No. 100 mesh screen, and also with the change in gradation, if any, due to changes in feed condition or load on the crusher.
2. Quality of the sand; that is, the shape of the individual particles or grains.
3. Amount of the circulating load, or that oversized portion of the crusher's product returned for further grinding.
4. Capacity of the crusher as a whole, and the capacity to produce sand passing a No. 4 mesh screen.

Description of plant.

In general, the plant lay-out consisted of the crusher, two belt conveyors, and a screen. These units were so arranged that the crusher could be operated in a closed circuit. The feed for the crusher was taken from a bin and consisted mostly of 1½- and ¾-inch stone. From the bottom of the storage bin the first conveyor transported crusher feed to the double deck screen located over the crusher. Material from the crusher was carried by the second conveyor and deposited on the first conveyor. In this manner new feed and crusher products were elevated together to the screen where by various chute arrangements the sized materials could be sent through the crusher or to the sand storage bin.

All oversize from the upper deck of the screen was dropped into the crusher while material passing the lower deck was chuted to the sand bin. That material retained on the lower deck was sent either to the crusher for further reduction or to storage as finished product. An adjustable blade gate at this point permitted separation in any desired proportion.

Gradation and particle shape.

The gradation of the product varied with the load on the crusher, the best gradation being obtained when the crusher was choke-fed. Figure 367, curve A, shows gradation for products under a choke-feed and curve B shows product with light load conditions (less than half capacity) for the same setting. Further investigation showed that the shape of grain varied with the load and the setting of the crusher. The best shaped particle was obtained when the crusher was set at about ⅜-inch opening and choke-fed. Smaller settings produced flat particles. The grain shape was not quite as cubical as the one produced by hammer mills. It had a greater proportion of wedge-shape particles than cubes. Figure 111 affords a comparison of the two products and clearly shows the preponderance of slag particles in the cone crusher product.

The plant was operated at full capacity under three conditions, differing only as to arrangement of screen cloths and the size of mesh used. A screen analysis curve of the products with each condition is shown as compared to a desired gradation curve in figure 367.

1. The screen was operated as a single deck screen with a cloth having a 0.194-inch opening.

2. In an effort to reduce the proportion of the larger sizes a screen cloth having a smaller opening (0.145 inch) was tried.

3. The first two tests indicated that a better sand could be obtained by the use of a double deck screen so arranged that the proportion of the coarser sizes is controllable to a degree by selective screening and recirculation. The upper deck was fitted with a cloth having a 0.194-inch opening, and the lower deck with one having an opening of 0.095 inch. The resultant product was composed of all sand passing the lower deck, plus one-third of that passing the upper screen. This condition gave the best results obtainable with the plant set-up.

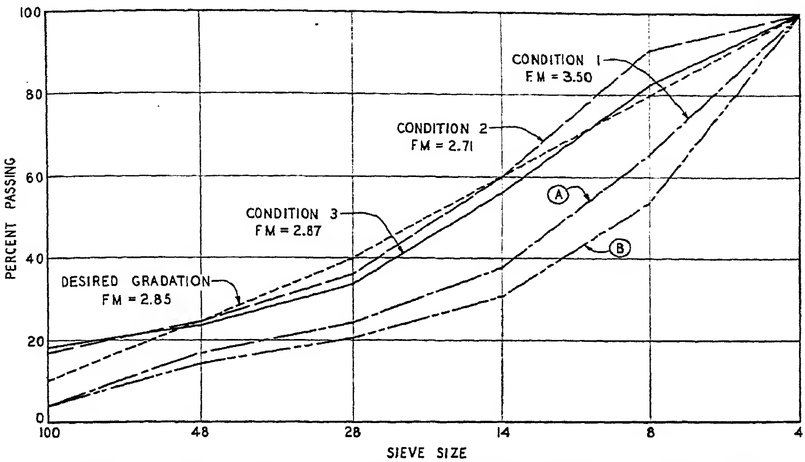


FIGURE 367.—*Sieve analysis of sand produced with a 3-foot Symons cone crusher.*

Crusher capacity.

Measurements were made of the crusher output by diverting the material on the conveyor belt into sample boxes for a noted time and converting to tons. The capacity for condition 1 above was 58 tons per hour at ⅜-inch setting with the feed consisting of one-third new feed and two-thirds circulating load. Under this condition the amount of material produced passing a No. 4 mesh screen was 20.5 tons per hour. For the best graded sand as in condition 3, the system output was 18.6 tons per hour.

Workability tests.

Tests were made on hammer mill and cone crusher sand to determine the amount of cement paste required to produce the same workability. The hammer mill sand was the product of a mill in a local cement plant, several truckloads of quarry stone having been hauled to the mill for crushing. Cone crusher sand was obtained from the experimental crushing plant.

Two methods were employed to determine the relative workability of the mortars made with the two sands. One method was the use of the flow table, and the other was the use of an improvised rig, called the "pendulumeter." This latter instrument consisted of a sharp blade fastened to a steel ball suspended so as to swing in a constant arc. The mortar to be tested was deposited in a trough, leveled, and placed in the path of the pendulumeter. The ball was pulled back to a fixed point and released. The point of the blade raked through the mortar for a certain distance, depending on the plasticity of the mortar.

Tests were made with mortar of different mixes but with constant water-cement ratio. Therefore, for two mixes of the same proportion of cement paste to aggregate, any difference in plasticity or workability would be due to the

nature of the sand in the mortar. Each sand had been screened and recombined to the same gradation.

Table 168 shows the result from the flow table and the pendulumeter tests for mortars mixed in varying proportions with a cement paste of a constant water-cement ratio (0.6 by weight).

TABLE 168.—*Results of flow table and pendulumeter tests*

Mix No.	Flow table tests		Pendulumeter tests	
	Cone crusher	Hammer mill	Cone crusher	Hammer mill
	Percent	*Percent*		
1	96.3	108.3	34.3	34.8
2	78.3	84.1	26.3	27.3
3	53.0	59.8	19.0	23.5
4	29.6	48.9	13.5	16.6

Mixes were then made using more cement paste with the cone crusher mortar than with the hammer mill sand in an effort to get the same percentage flow. These indicated that the same percentage flow could be obtained with the addition of about 4 to 5 percent more cement paste to the cone crusher sand mortar.

Further tests were made on the two sands using concrete instead of mortar. The sands were screened and recombined to the same gradation as before, and mixed with equal amounts of ¾-inch gravel of the same gradation. The batches were mixed with varying amounts of cement, using constant water-cement ratios. These tests substantiated the results from all other tests in that concrete made with sand produced by a cone crusher requires more cement for the same workability than concrete made with sand produced by a hammer mill.

Comparison of plant costs.

Capacity tests of the 3-foot cone crushers indicated that two 5½-foot machines would produce sand at the desired rate. These two machines with motors and accessories would cost about $36,000. Two hammer mills with equivalent capacity with motors and controls would cost about $12,000. The plant lay-out and other machinery units necessary to complete the installation for sand manufacture would be the same, whether cone crushers or hammer mills were used for the reduction units. The $24,000 difference in cost between the two installations when applied to the total sand requirement of 570,000 tons provided 4.2 cents per ton for hammer maintenance and repairs.

Summary of investigation.

Reviewing the points influencing the decision to use hammer mills instead of cone crushers to produce sand:

1. A cement saving of from 4 to 5 percent was indicated if hammer mill sand were used instead of cone crusher sand.

2. The lower first cost of the hammer mills would, in a large part if not wholly, pay for hammer renewals.

3. The gradations of the products of the two mills were substantially the same, indicating that for a better graded product selective screening was necessary. The material passing a No. 100 mesh sieve was expected to be somewhat more with the hammer mill product, resulting in additional waste through washing.

Appendix F

ACCEPTANCE TESTS

CEMENT ACCEPTANCE TESTS

Close plant control was required for the manufacture of modified type B portland cement as specified by the Authority. Inspectors were stationed at the mills furnishing cement and took samples of each 200 barrels under early contracts and of each 400 barrels under the later ones. These samples were sealed in airtight containers properly identified and shipped to the Knoxville laboratory for testing. Bins were sealed until the time of shipment or rejection as determined from the results of the tests.

At the time the samples were shipped by the plant inspector, a form letter was sent to the laboratory outlining pertinent facts concerning the shipment. At the laboratory, physical tests were made for soundness, time of initial and final set, strength (at 3, 7, and 28 days), and specific surface. Routine chemical and strength tests were made for each 800 barrels. Forms used for recording these tests are shown in figure 368. All routine acceptance tests were summarized on a single laboratory record (TV–472), which was further summarized (TVA–469) for distribution to the interested parties on the construction project to which the cement was consigned. Figure 369 shows these summary forms.

With the completion of all acceptance tests, the cement company and the plant inspector were advised by letter of the results of the tests. If the cement was acceptable the letter served as a release for shipment, and as a check of the shipments by the inspector.

Shipments of accepted cement from the mill to Norris Dam were made by railroad. Cars were loaded under the supervision of the inspector who executed the cement acceptance report and the daily report of shipment, shown in figure 369, making out a separate sheet for each silo. This latter report, accompanied by a certificate of inspection (a form letter) was sent to the materials division to complete the sequence of inspection routine necessary for the delivery of the cement.

STEEL ACCEPTANCE TESTS

The routine for the acceptance of materials other than cement usually required the physical inspection of the materials and workmanship at the plant of the manufacturer as explained in chapter 6. In cases where chemical analyses were required in addition to the physical tests, samples were submitted to the Knoxville laboratory.

Typical of these were the chemical analyses of the steel for the turbine shafts and crown plates. Borings from the former were analyzed for the percentage of manganese, phosphorus, and sulfur and from the latter for carbon, phosphorus, and sulfur.

Borings were taken by the TVA plant inspector, from test coupons and each sample sealed in a 3- by 5½-inch envelope for transmittal to the laboratory. The face of this envelope and the sample identification for accompanying it are shown in figure 370.

Upon the completion of the laboratory analyses the reports were completed as shown in figure 370, and the material was either accepted or rejected, depending on whether or not the material had met the specification requirements.

TURBINE ACCEPTANCE TESTS

Turbine acceptance tests, made during October 1937 on both power units, were conducted by means of the Gibson[1] method for measurement of water flow in closed conduits.

[1] Gibson, N. R., The Gibson Method, an Apparatus for Measuring the Flow of Water in Closed Conduits, Transactions, American Society of Mechanical Engineers, 1923. Thoma, D., Concerning the Degree of Accuracy of the Gibson Method of Measuring the

This method of water measurement is based upon the equation of impulse and momentum, and affords a means of determining the steady-state discharge in closed conduits by taking a continuous photographic record of pressure rise with respect to time, following fairly rapid closure of the turbine gates. With

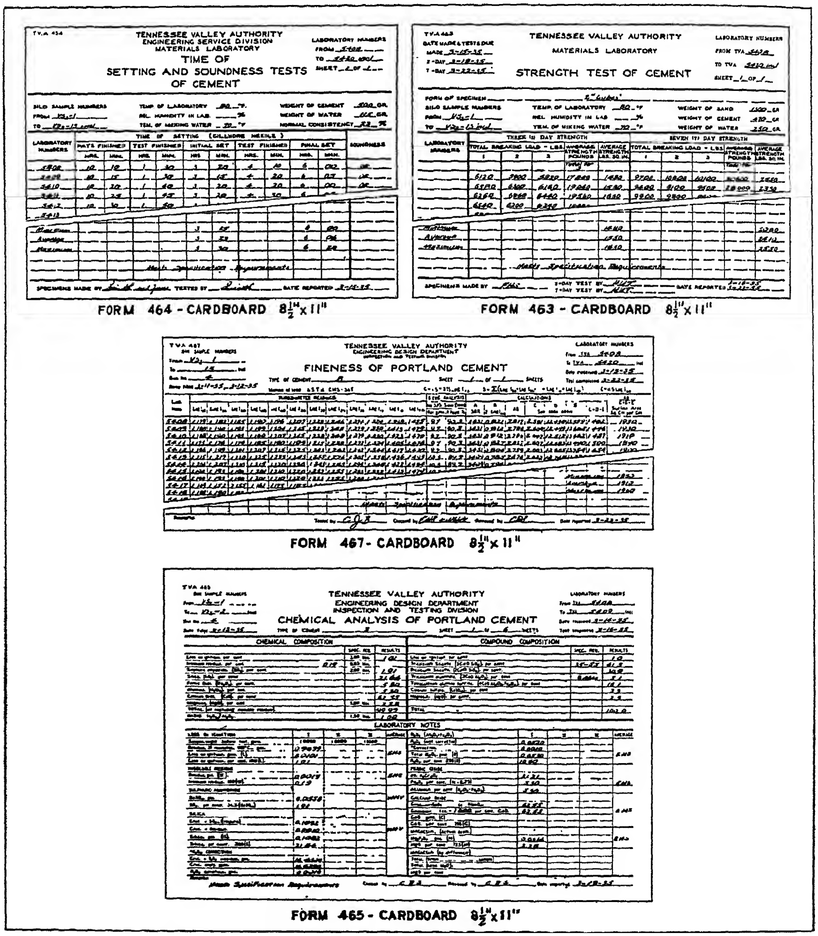


FIGURE 368.—*Forms used for routine acceptance tests on cement.*

the conduit dimensions known, the difference between the initial discharge and the leakage flow—which escapes when the turbine gates are completely closed—may be obtained from the pressure-time diagram. There are two alternate types of diagrams: the simple diagram in which pressure changes at one

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Flow of Water, Transactions, American Society of Mechanical Engineers, 1935, Hyd–57–4. Gibson, N. R., and Strowger, E. B., Experimental and Practical Experience with the Gibson Method of Water Measurement, Transactions, American Society of Mechanical Engineers, Hyd–57–5.

fixed cross section are recorded; and the differential diagram, in which the instantaneous difference between changes in pressure between two fixed cross sections in the conduit are recorded. The differential method was employed because of the slightly greater accuracy.

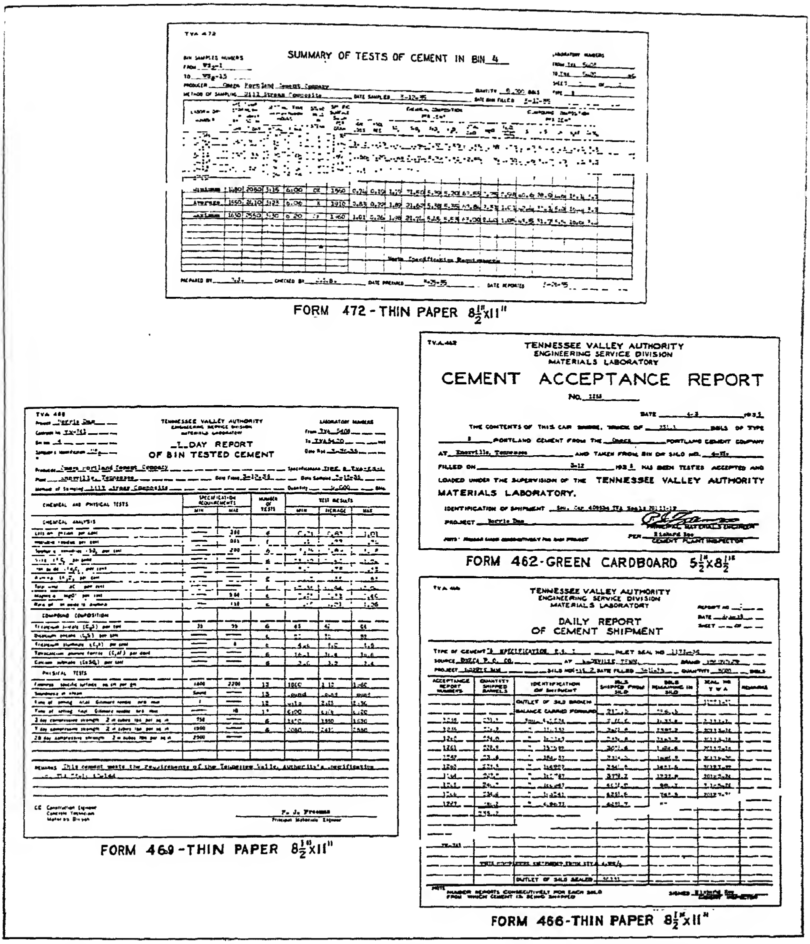


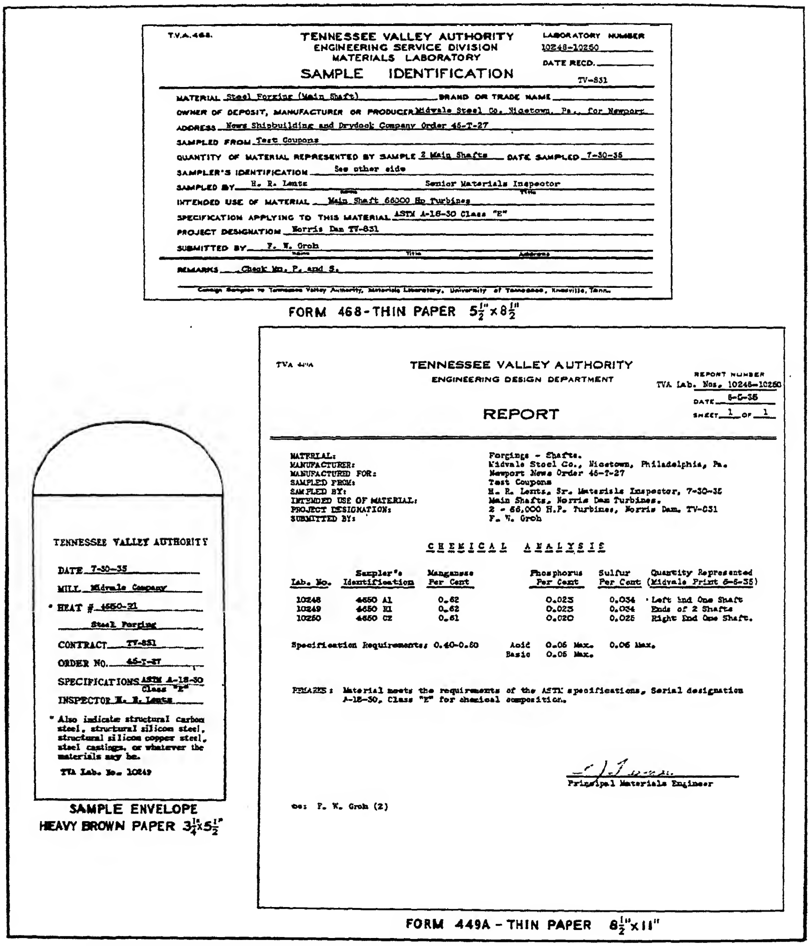
FIGURE 369.—*Forms used for summarizing cement tests and for reporting acceptance and shipment.*

Tests made.

The entire test was conducted under the provision of the Power Test Code of the American Society of Mechanical Engineers, Series 1926, except that all electrical instruments, including current and potential transformers, were not calibrated both before and after the test. Computation of turbine efficiencies required readings to determine the flow, net and gross heads, and power output at various gate openings. Readings on overspeed were also made.

Arrangement of equipment.

For the Gibson apparatus two sets of four piezometers were installed in each penstock, as shown in figure 371, at a distance of 91 feet 10 inches apart. Pipe connections from each set of taps terminated at the walkway at the downstream face of the dam where the Gibson pressure-recording apparatus was installed.



T.V.A. 468.

TENNESSEE VALLEY AUTHORITY
ENGINEERING SERVICE DIVISION
MATERIALS LABORATORY
SAMPLE IDENTIFICATION

LABORATORY NUMBER 10248-10250
DATE RECD. ____
TV-831

MATERIAL Steel Forging (Main Shaft) BRAND OR TRADE NAME ____
OWNER OF DEPOSIT, MANUFACTURER OR PRODUCER Midvale Steel Co. Nicetown, Pa., for Newport
ADDRESS News Shipbuilding and Drydock Company Order 46-T-27
SAMPLED FROM Test Coupons
QUANTITY OF MATERIAL REPRESENTED BY SAMPLE 2 Main Shafts DATE SAMPLED 7-30-35
SAMPLER'S IDENTIFICATION See other side
SAMPLED BY H. R. Lentz (Name) Senior Materials Inspector (Title)
INTENDED USE OF MATERIAL Main Shaft 66000 Hp Turbines
SPECIFICATION APPLYING TO THIS MATERIAL ASTM A-18-30 Class "E"
PROJECT DESIGNATION Norris Dam TV-831
SUBMITTED BY F. W. Groh (Name) (Title) (Address)
REMARKS Check Mn, P, and S.

Consign Samples to Tennessee Valley Authority, Materials Laboratory, University of Tennessee, Knoxville, Tenn.

FORM 468-THIN PAPER $5\frac{1}{2}'' \times 8\frac{1}{2}''$

TENNESSEE VALLEY AUTHORITY

DATE 7-30-35
MILL Midvale Company
* HEAT # 4650-21
Steel Forging
CONTRACT TV-831
ORDER NO. 46-T-27
SPECIFICATIONS ASTM A-18-30 Class "E"
INSPECTOR H. R. Lentz

* Also indicate structural carbon steel, structural silicon steel, structural silicon copper steel, steel castings, or whatever the materials may be.

TVA Lab. No. 10249

SAMPLE ENVELOPE
HEAVY BROWN PAPER $3\frac{1}{4}'' \times 5\frac{1}{2}''$

TVA 449A

TENNESSEE VALLEY AUTHORITY
ENGINEERING DESIGN DEPARTMENT
REPORT

REPORT NUMBER TVA Lab. Nos. 10248-10250
DATE 8-5-35
SHEET 1 OF 1

MATERIAL: Forgings - Shafts.
MANUFACTURER: Midvale Steel Co., Nicetown, Philadelphia, Pa.
MANUFACTURED FOR: Newport News Order 46-T-27
SAMPLED FROM: Test Coupons
SAMPLED BY: H. R. Lentz, Sr. Materials Inspector, 7-30-35
INTENDED USE OF MATERIAL: Main Shafts, Norris Dam Turbines.
PROJECT DESIGNATION: 2 - 66,000 H.P. Turbines, Norris Dam. TV-831
SUBMITTED BY: F. W. Groh

CHEMICAL ANALYSIS

Lab. No.	Sampler's Identification	Manganese Per Cent	Phosphorus Per Cent	Sulfur Per Cent	Quantity Represented (Midvale Print 6-6-35)
10248	4650 A1	0.62	0.023	0.034	Left End One Shaft
10249	4650 B1	0.62	0.023	0.034	Ends of 2 Shafts
10250	4650 C2	0.61	0.020	0.025	Right End One Shaft.

Specification Requirements: 0.40-0.60 Acid 0.06 Max. 0.06 Max.
Basic 0.05 Max.

REMARKS: Material meets the requirements of the ASTM specifications, Serial designation A-18-30, Class "E" for chemical composition.

Principal Materials Engineer

cc: F. W. Groh (2)

FORM 449A - THIN PAPER $8\frac{1}{2}'' \times 11''$

FIGURE 370.—*Sample envelope and form for steel acceptance tests.*

The gross head on the turbine was measured by reading the reservoir and tail water elevations. The tail water elevation was determined by means of a float gage, located in the powerhouse, which is a part of the regular operating equipment. Indication of the float gage level is transmitted to the control room by means of a standard selsyn motor arrangement.

Measurement of the net head on the turbine was made by means of a special water manometer constructed by the Authority. One leg of this manometer was connected to a pipe which led to four net head piezometers located a few feet downstream from the entrance to the scroll case. The other leg was connected to a pipe which led to piezometer No. 9, located in the reservoir on the upstream face of the dam. Compressed air, introduced in a header joining the top of both legs, depressed the water columns. The difference in height of the water columns represented the sum of the velocity head at the net head piezometer section, and the loss of head from the reservoir to the net head piezometer section.

Although not specifically required for determining turbine efficiency, an additional piezometer line was installed directly behind the rack structure and was used to determine the head loss due to the trashracks alone.

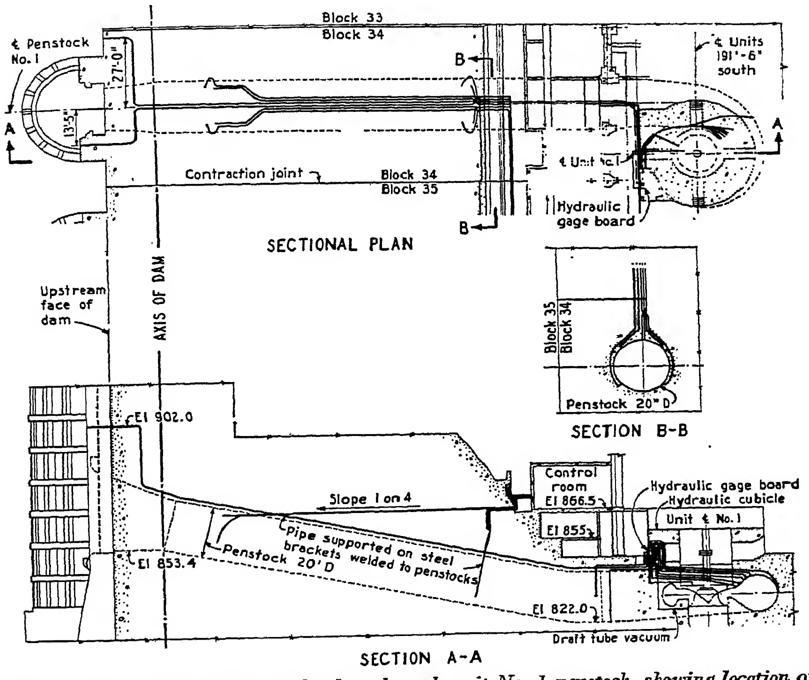


FIGURE 371.—*Section view of the dam through unit No. 1 penstock, showing location of piezometers and recording devices.*

The power output used in computing turbine efficiency was obtained by measuring the electrical output of the units with suitable correction for generator losses. These losses were determined from independent tests on the generators, the details of which are covered under that section of this appendix entitled "Generator Acceptance Tests." Power output of the generators was measured by two sets of single-phase wattmeters connected to calibrated standard-station current and potential transformers permanently connected to the generator leads, and these were added to obtain the official generator output. A check on the above readings was obtained by a polyphase wattmeter and a recording wattmeter. The output of the main and pilot exciters, aggregating about 85 kilowatts at full gate, were included with the main generator output in computing turbine performance. The power factor of the generator load was kept at unity throughout the turbine and combined unit tests.

Preliminary measurements.

To determine a pipe factor to be used in connection with the Gibson flow measurement, the final cross sections of the penstock pipes were measured at 10 different points. These were made at four different diameters at each of the 10 sections after the dam had been completed, the penstocks cleaned and painted, turbine and generator assembled, and all concrete poured. These 80 measurements averaged 19 feet 11¹³⁄₁₆ inches, and ranged between 19 feet 10½ inches and 20 feet 1⅛ inches.

Prior to the test, relative densities of mercury and water were computed from Smithsonian tables with due regard to the temperatures. To check the results obtained a manometer was set up in which a 127.80-foot water leg was balanced by a 9.405-foot mercury leg at a temperature of 14.5° C., resulting in a relative density of 13.5885. The mercury used in this test was a portion of that used in the actual tests.

To obtain the total turbine discharge for each test run, it was necessary to add the turbine leakage quantity to the Gibson diagram quantity. The leakage quantity for both turbines was computed from the measured clearances at the bottom, top, and between the turbine gates. The orifice area created by these clearances was computed to be 0.0991 square foot for each of the two units. Experience gained from making leakage tests on other turbines indicates that the coefficient of discharge "C" in the standard orifice formula is approximately unity (1.0) for this type of orifice. With these values and at a head of 170.70 feet on the orifice, the leakage quantity was computed to be 10.4 cubic feet per second. A value of 10.0 cubic feet per second was used for both units, which is approximately 0.22 percent of the turbine discharge (approximately 4,550 cubic feet per second) at full gate under a net head of 180.0 feet.

A governor closing time was chosen to give the most favorable adjustment to the recording apparatus, which was found to be about 15 seconds.

Test results.

Results of the turbine and combined unit efficiency tests for 180 feet net head are given in table 169. Figure 372 is a graphical presentation of the same data for unit No. 1. The results were practically the same for the two units. Although the acceptance tests were based on performance at a net head of 180 feet, the curves in figure 372 also present the test data on the basis of performance at a gross head of 180 feet.

TABLE 169.—*Results of turbine and unit efficiency tests*

	Unit of measure	Unit 1	Unit 2	Guarantee
Turbine efficiency at 60,000 horsepower	Percent	93.1	93.3	91.0
Maximum turbine efficiency	do	93.2	93.3	
Output at maximum efficiency	Horsepower	62,000	62,000	60,000
Generator output at maximum turbine efficiency.	Kilowatt	45,400	45,400	
Discharge at maximum turbine efficiency	Cubic feet per second	3,259	3,255	
Maximum combined turbine and generator test efficiency.	Percent	91.3	91.5	
Corresponding generator output	Kilowatt	44,000	44,000	
Turbine efficiency at 75,200 horsepower	Percent	88.9	89.9	86.0
Full gate test capacity:				
Turbine	Horsepower	78,300	78,300	75,200
Turbine efficiency	Percent	84.1	84.0	86.0
Generator	Kilowatt	57,400	57,400	
Turbine discharge	Cubic feet per second	4,560	4,562	
Guaranteed capacity	Percent	104	104	

The total head loss between the reservoir and turbine amounted to 0.61 foot at full turbine gate opening and 0.33 foot at best gate opening, indicating high over-all plant efficiency. The loss at the trashracks amounted to only 0.01 foot.

During the tests made to determine the runaway speed, unit No. 1 attained a speed of 188.8 revolutions per minute at 165.3 feet head, while unit No. 2 reached 184.0 revolutions per minute at 157.9 feet head, corresponding to a stepped-up value of 211 revolutions per minute at the maximum head of 207 feet, or 1.8 percent less than the maximum guaranteed runaway speed of 215

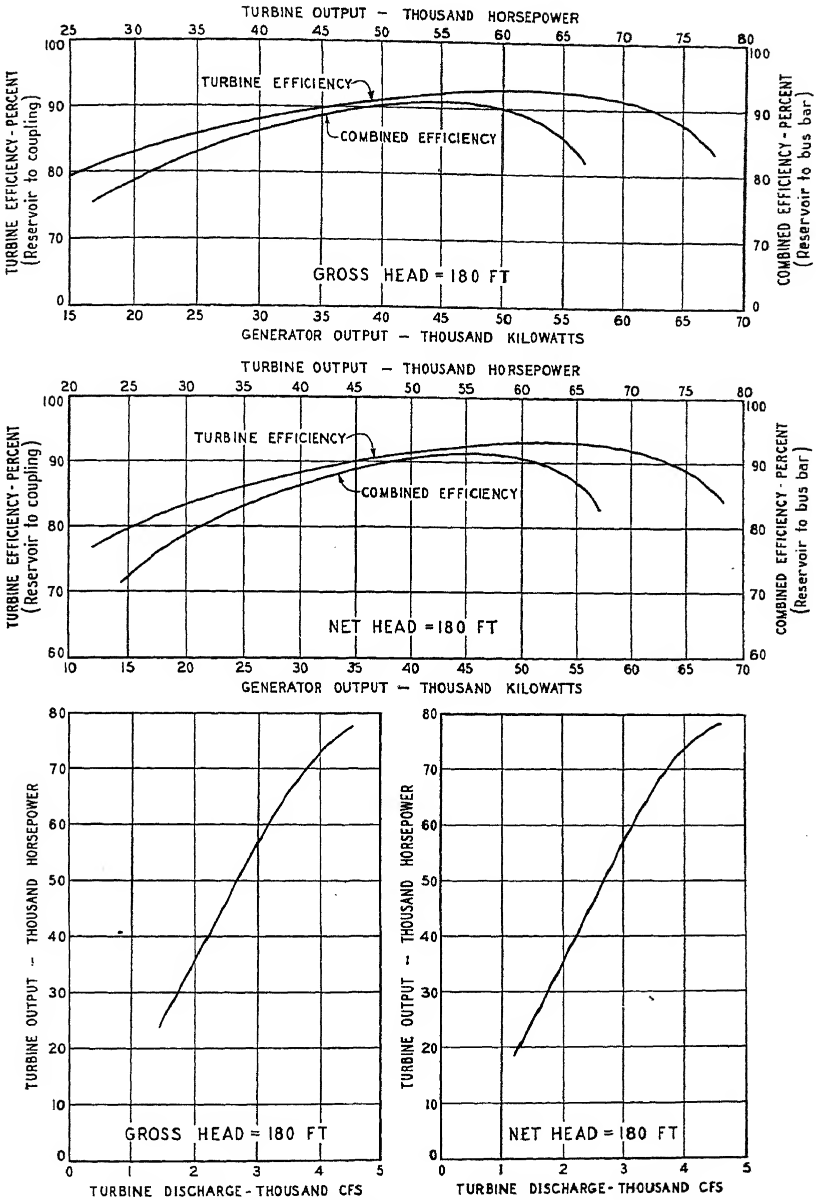


FIGURE 372.—*Results of efficiency test by Gibson method—Unit No. 1.*

revolutions per minute. Readings of the mercury manometers attached to the Winter-Kennedy scroll case taps indicated that the discharge is reduced materially under runaway conditions. Full gate discharge was approximately 25 percent less at runaway than at normal operating speed, while the corresponding half-gate discharge was reduced 12 percent.

GENERATOR ACCEPTANCE TESTS

The generator specifications required that the first seven of the following acceptance tests should be made on each generator, and that the balance should be made on only one generator.

1. Dielectric test of armature and field windings. Armature windings, 28,600 volts for 1 minute. Field windings, 5,000 volts for 1 minute.
2. Resistance of armature and field windings.
3. No load saturation test.
4. Short-circuit saturation test.
5. Telephone interference factor determination.
6. Overspeed test.
7. Short-circuit test at no load, rated speed, and 110 percent of rated voltage.
8. Efficiency test, including the determination of all losses.
9. Heat runs to determine temperature rise under continuous load at rated output.
10. Deviation of wave form factor.
11. Test to determine direct-axis transient reactance and synchronous reactance.
12. Test to determine short-circuit ratio.

These tests were completed in December 1937 with the exception of a slight revision of test No. 1 for generator No. 2 and omission of test No. 7 for both generators. The dielectric test for generator No. 2, which was the first one tested, resulted in a failure from the neutral leads to the bolts holding the supporting clamps in from 15 to 37 seconds. The manufacturer reinsulated the leads and retested at 21,500 volts by mutual agreement between the Authority and the manufacturer. Similar reinsulation was made on generator No. 1 and it was tested at 28,600 volts. The short-circuit test was waived for both generators.

Cold resistance readings of the armature and field windings were measured by the voltmeter-ammeter method. The resistances corrected to 75° C. are shown in table 170.

TABLE 170.—*Resistances corrected to 75° centigrade*

	Unit No. 1	Unit No. 2
Resistance in ohms:		
Rotor—Field	0.232010	0.233300
Stator—Phase BC	.022856	.022831
Stator—Phase AC	.022826	.022876
Stator—Phase AB	.022856	.022852

As the saturation curves for both units were almost identical, only those for unit No. 2 are shown in this appendix. Figure 373 shows saturation curves for both no-load and short-circuit conditions. Full-load saturation curves at 100, 90, and 0 per cent power factor are also included.

The telephone interference factor was measured with an interference meter and found to be:

	Unit No. 1	Unit No. 2
Telephone interference factor:		
Line to line	18.84	18.43
Residual	11.86	11.70

The overspeed test for each generator was made in conjunction with the overspeed test for the entire unit including the turbine and generator. The

results have previously been outlined in that part of this appendix covering the turbine acceptance tests.

The efficiency tests were made on unit No. 2 at 90- and 100-percent power factor for 25-, 50-, 75-, and 100-percent loads. The results of these tests, including all losses and efficiencies, are shown in table 171. Figure 374 shows graphically

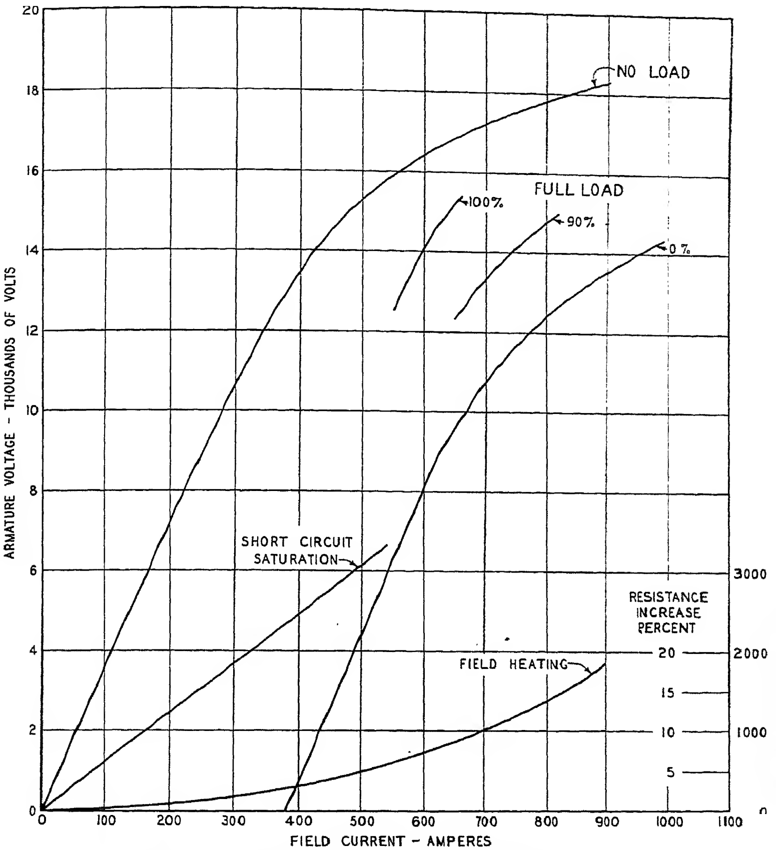


FIGURE 373.—*Saturation curves and field heating curve of unit No. 2.*

the same results. The determination of losses was made by the retardation method, which necessitated calculating the flywheel effect of the rotating parts. This amounted to:

$$W r^2 = 65{,}340{,}000 \text{ pounds-feet}^2$$

The heat run was made on unit No. 1 by loading the generator into the 154-kilovolt system. A load of 56,000 kilovolt-amperes at 0.76 power factor was obtained. Power measurements were taken with the switchboard instruments.

Field volts and amperes were read by means of portable instruments and the temperature of the field was calculated from the rise in resistance by means of the curve of figure 373. All eight coolers were in service and cooling water temperature was 13° C. Since the amount of cooling water could not be

measured, the main valve was throttled to give a temperature of not over 85° C on the machine. Temperature readings were taken of the stator windings, stator core, exhaust air, entering air, and guide and thrust bearings. The temperature of the field at the end of the test was 85.5° C. The ingoing air was at 34.5° C. The temperature rise was thus 50.5° C. for the armature and 51.0° C for the field. The field current was 888.1 amperes at 0.76 power factor during the test. At 0.9 power factor, the field amperes would have been 729 and the field temperature rise 30.5° C. The temperature rise of both armature and field were thus found to be within guarantee of 60° C. above 40° C. ingoing air with cooling water at 25° C.

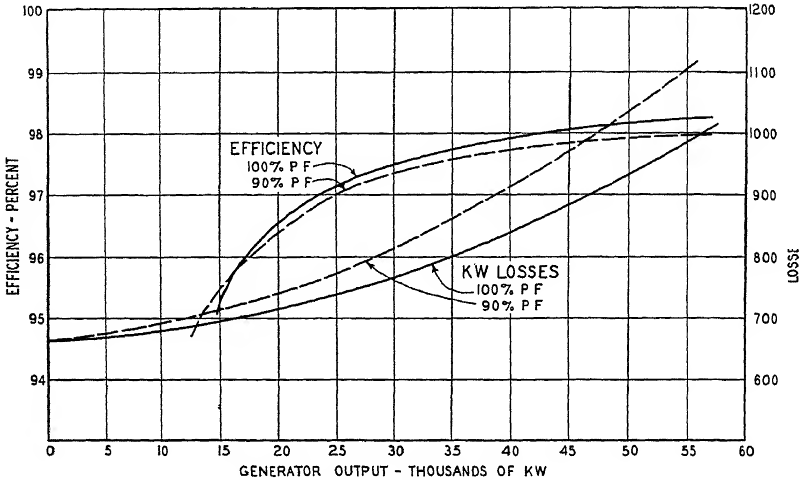


FIGURE 374.—*Losses and efficiencies of unit No. 2.*

TABLE 171.—*Acceptance tests—generator No. 2, tabulation of losses and efficiencies*

	90 percent power factor				100 percent power factor			
Percent load	100	75	50	25	100	75	50	25
Field amperes	729.00	650.00	571.00	496.00	590.00	546.00	502.00	460.00
LOSSES								
Friction and windage [1]	340.00	340.00	340.00	340.00	340.00	340.00	340.00	340.00
Core loss	276.00	275.00	274.50	274.00	276.00	275.00	274.50	274.00
Stator I^2R loss at 75° C	188.50	106.03	47.12	11.78	188.50	106.03	47.12	11.78
Load loss	98.00	55.00	25.00	6.50	98.00	55.00	25.00	6.50
Rotor I^2R loss at 75° C	123.98	98.57	76.07	57.39	81.20	69.25	58.79	49.37
Main exciter losses	11.30	9.09	7.20	5.57	7.68	6.656	5.72	4.855
Pilot exciter losses	7.422	6.887	6.354	5.823	6.620	6.089	5.558	5.032
Total losses	1,045.202	890.577	776.244	701.063	998.000	858.325	756.688	691.537
Kilowatt rating	50,400.000	37,800.000	25,200.000	12,600.000	56,000.000	42,000.000	28,000.000	14,000.000
Kilowatt rating+kilowatt losses	51,445.202	38,690.577	25,976.244	13,301.063	56,998.000	42,858.325	28,756.688	14,691.537
Percent losses	2.032	2.302	2.988	5.271	1.751	2.003	2.632	4.707
Percent efficiency:								
From test	97.968	97.698	97.012	94.729	98.249	97.997	97.368	95.293
Guarantee	97.300	97.000	96.100	93.400	97.750	97.450	96.600	94.100

[1] Includes friction and windage of generator, main and pilot exciters.

To determine the deviation factor of the wave form, an oscillogram was taken on each phase of unit No. 2, and one polar-oscillogram of the wave form was also taken. The wave forms shown on these oscillograms were considered satisfactory.

Reactance tests were also made on unit No. 2 running at no load, rated speed, and one-third voltage. The constants determined from oscillograms of the three-phase current, field amperes, and timing wave were:

	Percent
Synchronous reactance	97.5
Subtransient reactance	23.2
Transient reactance	34.4

The short-circuit ratio was computed to be 1.09 with values taken from the no-load and short-circuit saturation curves to form this ratio:

$$\frac{\text{Field amperes at no-load rated voltage}}{\text{Field amperes at rated armature amperes, armature short circuited}}$$

APPENDIX G

TYPICAL SPECIFICATION FORMS

Formal notices to bidders specifying the material or services needed and describing the requirements and conditions of the bidding as specified in section 9 (*b*) of the amended act for purchasing major equipment or materials are shown on the following pages. These forms were modified slightly from time to time as conditions required, but the ones shown, together with the specifications for the discharge conduit slide gates are typical for formal contracts. A purchase order form for an informal contract is shown in figure 285.

TVA 364

TENNESSEE VALLEY AUTHORITY

Date ____________________, 193 .

NOTICE TO BIDDERS

Sealed bids, in ____________________, will be received at the office of the Director of Purchases until ________ m., central standard time, ________________, 193 ; for ____

Invitation to bid, conditions, bid bond, specifications, form of contract, and performance bond may be obtained on or after ____________________, 193 , at the office of the Director of Purchases. A charge of $ __________, which will not be refunded, will be made for the above documents.

C. H. GARITY,
Director of Purchases.

Address:
OLD POST OFFICE,
Knoxville, Tenn.

TVA 356

TENNESSEE VALLEY AUTHORITY

Date ____________________, 193 .

INVITATION TO BID

Sealed bids, in ____________________, subject to the conditions set forth herein, will be received at the office of the Director of Purchases until __________ o'clock, ____ M., central standard time ____________________, 193 , and then publicly opened for ____

____

Bids must be submitted on the Authority's form of bid enclosed herewith. Envelopes containing bids must be sealed and marked on the upper left-hand corner with the name and address of the bidder and the date and hour of the opening and sent to the address set forth below (use enclosed gummed label for this purpose).

C. H. GARITY,
Director of Purchases.

Address:
OLD POST OFFICE,
Knoxville, Tenn.

155813—40——50

CONDITIONS OF BID

1. *Bid Documents.*—These Conditions, the forms of Bid Bond and Performance Bond, the Specifications, Contract, and other documents attached hereto, shall constitute a part of the Bid and Contract, and should be carefully examined by the bidder before submitting bid. Bids not conforming to these conditions will be subject to rejection.

2. *Information Required of Bidders.*—Bidder shall supply all information called for in the Bid Schedules, and Specifications, and these Conditions.

3. *Supporting Data.*—Each bidder shall submit a full set of all supporting letters, drawings, specifications, and other data with each copy of his bid. Within five (5) days after date of notice of award, the Contractor shall furnish fifteen (15) additional copies of all drawings, specifications, etc., which were submitted with his bid to the Director of Purchases, Tennessee Valley Authority, Old Post Office Building, Knoxville, Tenn.

4. *Bid Form.*—All bids must be based upon the specifications and must be made upon the blank forms of proposal which are hereto attached. The form of proposal must not be changed; all appropriate entries of bid price, time of shipment, weight, and other data shall be made in the several blank spaces provided therefor. Proposals must give the price for each item of the work proposed both in words and in figures.

5. *Bid Guaranty.*—Each bidder shall furnish a guaranty in the form of a Bid Bond in the form attached, with security satisfactory to the Authority, or a certified check on a bank that is a member of the Federal Reserve System, to the order of the Authority, in the amount of not less than five percent (5%) of the total amount of all lots covered by his bid.

6. *Weights and Freight Charges.*—Each bidder shall state, in the blanks provided therefore in the Schedule, the total shipping weight of each item he proposes to furnish, and these stated weights will be used in computing the delivered cost to the Authority.

7. *Prices.*—Prices should be stated in units of quantity specified with packing included. In case of discrepancy between words and figures in a bid the prices written in words will govern.

8. *Trade Names.*—If any item, or essential part of any item, bid upon has a trade name or brand, such trade name or brand must be stated in the bid.

9. *Federal Tax.*—The prices bid in the schedules shall include any Federal Tax heretofore imposed by the Congress which is applicable to the work under this contract. If any sales tax, processing tax, adjustment charge, or other tax or charge is imposed or changed by the Congress after the date set for opening bids and made applicable directly upon the production, manufacture, or sale of the supplies to be furnished and is paid by the Contractor on the articles or supplies furnished, then prices stated in the schedules will be increased or decreased accordingly, and any amount due the Contractor as a result of such change will be charged to the Authority and should be entered on vouchers (or invoices) as separate items.

10. *Experience and Facilities.*—The bidder may be required to furnish satisfactory evidence of experience and facilities for handling the work, and may also be required to furnish satisfactory evidence that he has, in the judgment of the Authority, adequate shops, plants, facilities, equipment, financial resources, business and technical organizations, and working capital to begin the work promptly and prosecute it vigorously in such manner as to secure completion within the time specified.

11. *Samples.*—Samples of material and manufactured articles, when required, must be furnished free of expense prior to the openings of bids, and, if not destroyed, will, upon request, be returned at the bidder's expense.

12. *Shipment on Government Bills of Lading.*—At the request of the Authority the Contractor shall make any rail shipment on Government Bills of Lading furnished by the Authority rather than Commercial Bills of Lading, unless bidder has stated in the place provided in the Schedule that bid is based on making rail shipments on Commercial Bills of Lading. The Authority will advise the Contractor at the time of award whether shipment(s) are to be made on Government Bills of Lading or on Commercial Bills of Lading. If shipment(s) are required on Government Bills of Lading, title to material shall pass to the Authority, f. o. b. cars at point(s) of origin, anything to the contrary herein contained notwithstanding. Such passing of title f. o. b. cars at point(s) of

origin shall not relieve the Contractor of any obligations assumed or retained by him. There is also reserved the right to make inspection at destination, and to reject any and all material found not to comply with the contract, title to such material to revert to the Contractor and all freight charges on such rejected material whether shipment was made on Government Bills of Lading or on Commercial Bills of Lading shall be paid by the Contractor.

If award is made on prices quoted f. o. b. destination and shipment(s) are required on Government Bills of Lading, the Authority shall be entitled to deduct and retain out of any payments due the Contractor a sum equal to the full commercial freight.

13. *Evaluation of Bids.*—Whenever applicable, equalizing elements or factors not specifically mentioned or provided for herein, such as the cost of transportation, erection, construction, or of inspection (including salaries, travel, and subsistence expenses), or any other element or factor in addition to that of the bid price which would affect the final cost to the Authority may be taken into consideration in making award of contract.

In comparing bids and in making awards the Authority may consider such factors as relative quality and adaptability of supplies or services, the bidder's financial responsibility, skill, experience, record of integrity in dealing, ability to furnish repairs and maintenance services, the time of delivery or performance offered, and whether the bidder has complied with the specifications.

14. *Adjournment.*—The Authority reserves the right to adjourn the time of opening bids from time to time by notice before or at the stated time of opening.

15. *Award and Rejection.*—The Authority reserves the right to reject any and all bids, to waive any informality in the bid, and, to accept or reject one or more of the items in the bid. Award will be made to only one bidder on this Invitation.

16. *Discounts.*—Bidder may offer discounts in the place provided in the bid and the discount periods in such event shall be computed from receipt by the Authority of properly certified invoices, in triplicate, as stated in Article 4 of the contract and receipt at destination of material covered by said invoices. Discounts offered shall be applied to any payment made within such discount period.

17. *Contract, Performance, and Payment Bonds.*—The bidder to whom award is made shall execute the Authority's form of Contract, Performance Bond, with security satisfactory to the Authority, within ten (10) days after the prescribed forms are presented for signature. These forms should not be executed prior to award of contract.

18. *Time of Completion.*—Time of completion will be of the essence of the contract. Work under this contract shall be completed as set out in Article 5 of the attached Contract.

19. *Bidder's Specifications and Drawings.*—Each bidder shall submit with his proposal, complete specifications, cuts, and drawings describing and illustrating the equipment which he proposes to furnish in sufficient detail to make possible a complete understanding of the equipment on which the bid is based.

If the bidder intends to use, in the assembly of the equipment offered, parts purchased from another manufacturer, he shall state the name of such other manufacturer, or manufacturers, and shall give detailed information on all items purchased.

20. *Authority's Wage Scale.*—Attention of the bidders is called to the fact that all laborers and mechanics engaged in carrying out this contract on premises under the control of the Authority shall be paid not less than the rates of wages set forth in the attached Wage Scale, and the Contractor shall comply to all of its provisions.

21. *Comparable Installations.*—Each bidder shall submit with his bid a list of installations of equipment he has made, comparable to the equipment he proposes to furnish under this Invitation. Such list should include size of installation, by whom designed, for whom installed, date, and place of installation.

TVA 358 (2-15-37)

TENNESSEE VALLEY AUTHORITY

FORM OF BID

Place____________________
Date ____________________

To: TENNESSEE VALLEY AUTHORITY,
Knoxville, Tenn.

In compliance with your invitation for bids to furnish materials, supplies, and/or perform all work required, listed on the accompanying schedules, in strict accordance with all documents which will constitute the contract,
the undersigned,____,
a corporation organized and existing under the laws of the State of____________,
a partnership consisting of______________________________________,
an individual trading as____
of the city of____,
hereby proposes to furnish, within the time specified, the materials, supplies, and/or perform all work required at the prices stated opposite the respective items listed on the schedules and agrees, upon receipt of written notice of the acceptance of this bid within__________days (60 days if no shorter period be specified) after the date of opening of the bids, to execute, if required, the Authority's form of contract in accordance with the bid, as accepted, and to give bond, if required, with good and sufficient surety or sureties, satisfactory to the Authority, for the faithful performance of the contract, within ten (10) days after the prescribed forms are presented for signature. Discounts will be deducted from the gross amount of the contract price, unless otherwise specified by the bidder on this Form.

Discount will be allowed for prompt payment as follows: 10 calendar days __________per centum (%); 20 calendar days__________per centum (%); 30 calendar days__________per centum (%); or as stated in the schedules.

____
(Witness to signature)

____
(Full name of builder)

____
____
(Address)

TVA 361

TENNESSEE VALLEY AUTHORITY

SCHEDULE FOR FORM 360

ITEM No.	ARTICLES OR SERVICES	QUANTITY	UNIT	UNIT PRICE	AMOUNT	
					Dollars	Cents

(Bidder)

IDENTIFICATION OF BIDDER'S SPECIFICATIONS AND DRAWINGS

The foregoing proposal is based on furnishing equipment in accordance with the Authority's specifications here following, and also with the undersigned bidder's supplementary specifications and drawings which are transmitted herewith, and which are marked, signed, and dated for identification as follows:

Bidder

TVA 359

TENNESSEE VALLEY AUTHORITY

Bid Bond

Know All Men by These Presents, That we, ________________________________
as principal, and ____, as surety, are held and firmly bound unto the Tennessee Valley Authority, hereinafter called the Authority, in the penal sum of ____________________________ dollars ($______________) lawful money of the United States, for the payment of which sum well and truly to be made, we bind ourselves, our heirs, executors, administrators, and successors, jointly and severally, firmly by these presents.

The Condition of This Obligation is Such, That whereas the principal has submitted the accompanying bid, dated ______________________________, 193 , for

Now Therefore, If the principal shall not withdraw said bid within the period specified therein after the opening of the same, or, if no period be specified, within sixty (60) days after said opening, and shall within the period specified therefor, or, if no period be specified, within ten (10) days after the prescribed forms are presented to him for signature, enter into a written contract with the Authority, in accordance with the bid as accepted, and give bond with good and sufficient surety or sureties, as may be required, for the faithful performance and proper fulfillment of such contract, or in the event of the withdrawal of said bid within the period specified, or the failure to enter into such contract and give such bond within the time specified, if the principal shall pay the Authority the difference between the amount specified in said bid and the amount for which the Authority may procure the required work and/or supplies, if the latter amount be in excess of the former, then the above obligation shall be void and of no effect, otherwise to remain in full force and virtue.

In Witness Whereof, The above-bounded parties have executed this instrument under their several seals this ______ day of __________________________, 193 , the name and corporate seal of each corporate party being hereto affixed and these present duly signed by its undersigned representative, pursuant to authority of its governing body.

[SEAL] ______________________________
Principal.
Address ______________________
____

Attest:
______________________________ By ________________________
(Title)

[SEAL] ______________________________
Surety.
Address ______________________
____

Attest:
______________________________ By ________________________
(Title)

The rate of premium on this bond is $__________ per thousand.
Total amount of premium charges, $__________

(The above must be filled in by corporate surety)

Contract No. TV-

TVA 362 (2–38)

TENNESSEE VALLEY AUTHORITY

Contract
for

Agreement, made this __________________ day of __________________, 193_ between Tennessee Valley Authority, hereinafter called "Authority," and___
a ____
of ____, hereinafter called "Contractor":

WITNESSETH

In consideration of the mutual covenants hereinafter stated, the parties agree for themselves, their personal representatives, successors, and ---- follows:

Art. 1. *Scope of Contract.*—

Art. 2. *Definitions.*—Whenever the words defined in this article, or pronouns used in their stead, occur in this contract, they shall have the meanings here given:

The words "Contracting Officer" shall mean the Director of Purchases of the Authority, acting directly or through his properly authorized agents.

The word "Engineer" shall mean the Chief Engineer of the Authority, acting directly or through his properly authorized agents.

The word "Contract" shall mean, collectively, all the covenants, terms, and stipulations in these articles of agreement and in the supplementary documents hereto attached which constitute essential parts of the agreement and are hereby made such parts thereof, to wit:

Invitation to Bid.

Conditions of Bid.

Contractor's Bid including Schedules.

Specifications and Drawings enumerated therein.

The word "Specifications" shall mean, collectively, all of the terms and stipulations contained in the general and detail specifications appended to this Agreement.

Art. 3. *Consideration.*—The Authority agrees to pay and the Contractor to accept as full payment for the complete performance of this contract, in accordance with all the terms and conditions herein contained, the respective sums stated opposite each item or lot of equipment in the schedules attached hereto, amounting to the total sum of ---------------- dollars ($----------).

For services of Erecting Engineers, if required, the Authority shall pay

----

dollars ($------------) each per day.

Art. 4. *Terms of Payment.*—Payments less such discounts and deductions as the Authority may be entitled to retain hereunder will be made as follows:

Eighty-five per cent (85%) of the contract price will be paid within thirty (30) calendar days after delivery at destination of all apparatus included in this contract, and receipt by the Authority at -------------------- of properly certified invoices.

The remaining fifteen per cent (15%) of the contract price will be paid within -------------------- calendar days after acceptance by the Authority of all apparatus included in this contract and receipt of pertinent invoices as above, or within -------------------- calendar days after receipt of all apparatus at destination and receipt of invoices as above whichever date may be the earlier, but such final payment shall not relieve the Contractor of any of his obligations under this contract and may be withheld until the Contractor has fully complied with all the terms of this contract on his part to be performed.

Separate invoices must be submitted in triplicate for each payment.

Discounts will be allowed for prompt payment as follows:

Ten (10) calendar days -------------------- percent (----------%); twenty (20) calendar days -------------------- percent (----------%); thirty (30) calendar days -------------------- percent (----------%); in accordance with Conditions of Bid.

Art. 5. *Commencement, Completion, and Delivery.*—Time is of the essence of this contract. Shipment from point of origin of all apparatus included in this contract shall be made within -------------------- calendar days after date of notice of award of contract, or not later than -------------------- 193--.

If award is made under Item 1a the Contractor shall start the work of installation on or before a date to be established by a written notice from the Authority, which shall be given not later than --------------------; provided that the date so established shall be not less than -------------------- calendar days later than the date of the written notice, and provided further, that the date so established shall not be earlier than -------------------- calendar days after the date of notice of award of the contract. The contractor shall complete the installation within -------------------- calendar days after the date so established for starting the work of installation.

ART. 6. *Delays and Remedies.*—If the Contractor refuses or fails to complete the work within the time specified in Article 5 hereof, or any extensions thereof, or if he assigns or encumbers any interest in this contract without prior written consent of the Authority, or makes an assignment for the benefit of creditors, or is thrown into receivership or adjudicated a bankrupt, or takes advantage of any bankruptcy or insolvency law, or refuses or fails to prosecute the work, or any separable part thereof, with such diligence as, in the opinion of the Authority, will insure its completion within the time specified therefor, or any extensions thereof, or fails to carry out any of his other obligations after written notice from the Authority so to do, the Authority, by written notice to the Contractor, may terminate the Contractor's right to proceed with performance of all or any part of the contract, may prosecute the same to completion by contract or otherwise, and may purchase similar materials or supplies in the open market or secure their manufacture and delivery by contract or otherwise: *Provided,* That the right of the Contractor to proceed shall not be terminated because of any delays in performance due to unforeseeable causes beyond his control and without his fault, such as acts of God or of the public enemy, acts of the Authority, fires, floods, epidemics, quarantines, strikes, freight embargoes, unusually severe weather, or delays of subcontractors due to such causes: *Provided further,* That the Contractor shall within ten (10) days from the beginning of any such delay notify the Contracting Officer in writing of the causes of delay. The Contracting Officer shall thereupon ascertain the fact and the extent of the delay, and his findings of fact thereon shall be final.

ART. 7. *Changes.*—The Contracting Officer may at any time, by written order, and without notice to the sureties, make changes in the drawings and specifications and within the general scope of the contract. If such changes cause an increase or decrease in the amount due under this contract, or in the time required for its performance, an adjustment will be made based on computations made on the basis of unit prices, if any, or in the percentage of the changes to the original scope of the work; and the contract shall be modified in writing accordingly. All claims for adjustment under this Article must be asserted within ten (10) days from the date the change is ordered, and in the meantime the Contractor shall proceed with the work so changed.

ART. 8. *Extras.*—No charge for extras will be allowed unless ordered in writing by the Contracting Officer and the price stated in such order.

ART. 9. *Failure of Congress to Appropriate Funds.*—If the operations of this contract extend beyond the current fiscal year, it is understood that the contract is made contingent upon Congress making the necessary appropriation for expenditures thereunder after such current fiscal year has expired. In case such appropriation as may be necessary to carry out this contract is not made, the Contractor hereby releases the Authority from all liability due to the failure of Congress to make such appropriation.

ART. 10. *Inspection.*—All materials and workmanship shall be subject to inspection and test at all times and places and, when practicable, during manufacture or construction. The Authority shall have the right to reject articles or work which contain defective materials or workmanship, or which do not comply with the specifications, and to require their replacement or correction in accordance with the contract, by and at the expense of the Contractor promptly after notification of rejection. If the inspection or test is made on the premises of the Contractor, the Contractor shall furnish, without additional charge, all reasonable facilities and assistance therefor.

Inspection and acceptance shall not be conclusive as regards latent defects, fraud, or such gross mistakes as amount to fraud. Inspection and acceptance or rejection of the material or work shall be made as promptly as practicable, but failure to inspect and accept or reject materials or work shall not impose liability on the Authority for such materials or work as are not in accordance with the specifications. No inspection, or anything disclosed thereby, shall affect any warranty of the Contractor. Reference is made to the specifications for further provisions, if any, as to inspection.

ART. 11. *Specifications and Drawings.*—Anything mentioned in the specifications and not shown in the drawings, or shown in the drawings and not mentioned in the specifications shall be of like effect, as if shown or mentioned in both. In case of differences between the drawings and the specifications, the specifications shall govern, except to the extent that the Engineer before the starting of the work affected by such differences shall have notified the Contractor that the drawings shall govern. Should any conflict arise

between the Authority's specifications hereto attached and those submitted by the Contractor, the former shall rule in all essential requirements, and, when not otherwise determined by the Engineer, in matters of detail also. In case of discrepancy in the specifications and drawings, the matter shall be immediately referred to the Engineer, without whose decision such discrepancies shall not be adjusted by the Contractor, and the Contractor shall not proceed with the work so affected until he has received written orders from the Engineer, approved by the Contracting Officer. The Authority will furnish, from time to time, such detail drawings and other information as it may consider necessary, unless otherwise provided.

ART. 12. *Responsibility for Materials and Work.*—Subject to the provisions of the paragraph of Conditions of Bid, entitled "Shipment on Government Bills of Lading," as to all materials and work covered by this contract, the risk of loss shall be upon the Contractor until delivery at the designated delivery point and inspection and acceptance thereof by the Authority. The Contractor shall further bear all risk on articles, materials, or work rejected pursuant to the terms of this contract. Where inspection is at point of origin but delivery by the Contractor is at some other point, the Contractor's responsibility shall continue until delivery is accomplished.

All materials and work covered by partial payments made shall thereupon become the sole property of the Authority, but this provision shall not be construed as relieving the Contractor from the sole responsibility for the care and protection of materials and work upon which payments have been made or the restoration of any damaged work until delivery is accomplished, or as a waiver of the right of the Authority to require the fulfillment of all of the terms of the contract.

ART. 13. *Disputes.*—Except as otherwise specifically provided in this contract, all disputes concerning questions of fact arising under this contract shall be decided by the Contracting Officer or his duly authorized representative, whose decision shall be final and conclusive upon the parties hereto as to such questions of fact. In the meantime the Contractor shall diligently proceed with the work as directed.

ART. 14. *Domestic Materials.*—Unless otherwise specified, it is understood and agreed that only such unmanufactured articles, materials, and supplies as have been mined or produced in the United States, and only such manufactured articles, materials, and supplies as have been manufactured in the United States substantially all from articles, materials, or supplies mined, produced, or manufactured, as the case may be, in the United States, shall be delivered or used pursuant to this contract.

The foregoing provision shall not apply to such articles, materials, or supplies of the class or kind to be used, or such articles, materials, or supplies from which they are manufactured, as are not mined, produced, or manufactured, as the case may be, in the United States in sufficient and reasonably available commercial quantities and of a satisfactory quality, or to such articles, materials or supplies as may be excepted by the Authority, under the proviso of title III, Section 3, of the Act of Congress approved March 3, 1933 (U. S. Code, Title 41, sec. 10b).

ART. 15. *Prevailing Wage Rate.*—Any laborers and mechanics employed in the construction, alteration, maintenance, or repair of buildings, dams, locks, or other projects shall be paid not less than the rate of wages for work of similar nature prevailing in the vicinity. In the event any dispute arises as to what are the prevailing rates of wages, the question shall be referred to the Secretary of Labor for determination, and his decision shall be final. In the determination of such prevailing rate or rates due regard shall be given to those rates which have been secured through collective agreement by representatives of employers or employees. The wage scale, if any, attached hereto represents in the opinion of the Authority the present prevailing rate of wages for the different classifications set forth therein, and in the absence of a decision of the Secretary of Labor as set forth above changing the same, shall be deemed by all parties hereto to be the minimum rates of wages that shall be paid by the Contractor for the work under this contract, insofar as applicable, and the Contractor shall comply with all regulations therein contained. The Authority may withhold from any monies due the Contractor any sum necessary to make up the full amount of wages due under this paragraph and may distribute it direct to those entitled thereto hereunder.

ART. 16. *Patents and/or Copyrights.*—The Contractor shall hold and save

the Authority, its officers, agents, servants, and employees harmless from liability of any nature or kind, including costs and expenses, for or on account of any trade-mark, copyrighted or uncopyrighted composition, secret process, patented or unpatented invention, article, or appliance manufactured or used in the performance of this contract, including their use by the Authority.

ART. 17. *Claims for Labor and Materials.*—The Contractor shall at his own expense assume the defense of and save harmless the Authority from all claims for materials furnished or work done; shall promptly discharge the same and not suffer any mechanics or other liens to remain outstanding against any of the property used in connection with the work; and shall, on request, furnish satisfactory evidence that all persons who have done work or furnished materials have been fully paid. If the Contractor fails to comply with his obligations in this respect, the Authority may take such steps as it may deem appropriate to discharge such liens or claims and may withhold from any monies due the Contractor such amount as may be necessary to satisfy and discharge any such claims and any cost and expense incident thereto.

ART. 18. *Officials not to Benefit.*—No Member of, or Delegate to, Congress or Resident Commissioner, or any officer or agent of the Authority shall be admitted to any share or part of this contract or to any benefit that may arise therefrom, but this provision shall not be construed to extend to this contract if made with a corporation for its general benefit.

ART. 19. *Additional Security.*—Should any surety upon the bond for the performance of this contract become unacceptable, the Contractor shall promptly furnish additional security as may be required from time to time to protect the interests of the Authority and of persons supplying labor or materials in the prosecution of the work contemplated by the contract.

ART. 20. *Contingent Fees.*—The Contractor warrants that he has not employed any person to solicit or secure this contract upon any agreement for a commission, percentage, brokerage, or contingent fee. Breach of this warranty shall give the Authority the right to annul the contract, or, in its discretion, to deduct from the contract price or consideration the amount of such commission, percentage, brokerage, or contingent fees. This warranty shall not apply to commission payable by contractors upon contracts or sales secured or made through bona fide established commercial or selling agencies maintained by the Contractor for the purpose of securing business.

ART. 21. *Assignment.*—This contract or any interest therein or in any monies due or to become due thereunder shall not be assigned, hypothecated, or otherwise disposed of without the previous consent in writing of the Contracting Officer. The Contractor shall on request file with the Authority copies of all subcontracts and terms of all commitments with subcontractors, and the Authority shall have the right to disapprove any thereof within five (5) days after receipt of such information.

ART. 22. *Indemnity.*—The Contractor shall be considered an independent Contractor for all the purposes of this contract, and all persons engaged in carrying out any of the Contractor's obligations hereunder shall be the servants of the Contractor or his subcontractors and not the servants or agents of the Authority.

The Contractor shall indemnify the Authority, its agents and servants, against any liability whatsoever resulting from the Contractor's operations under this contract; shall, at his own expense, assume the defense of all claims and actions for damages arising out of this contract which may be brought against the Authority by third persons; shall pay all judgments that may be rendered on such actions; and shall, if required by the Authority, carry insurance of character, form, and in amount satisfactory to the Authority, so protecting it.

The Authority reserves the right, also, to withhold from any sums due the Contractor, sufficient funds to satisfy such claims, and to adjust and to pay the same, upon a fair and reasonable basis, out of such funds so withheld, should the Contractor, after written notice from the Authority, fail to satisfy the same promptly: *Provided, However,* That the Authority shall not pay such claims should the Contractor submit the venue and jurisdiction of any court in which the Authority may be sued thereon.

ART. 23. *Waivers.*—No waiver of any breach of this contract shall be held to be a waiver of any other or subsequent breach. All remedies afforded the Authority in this contract shall be taken and construed as cumulative; that is, in addition to every other remedy provided herein or by law.

ART. 24. *Regulations.*—The Contractor shall conform to all local, State, and Federal statutes, ordinances, and regulations, and shall secure and pay for all necessary permits.

ART. 25. *Correspondence.*—Copies of all correspondence relating to work under this contract shall be furnished to the Director of Purchases.

ART. 26. *Claims and Protests.*—If the Contractor takes exception to any ruling or measurement of the Authority, he shall, within ten (10) days thereafter, file a formal written protest with the Director of Purchases, or be considered as having waived all future claims on account of ruling or measurement excepted to.

ART. 27. *Guaranty.*—The Contractor warrants for itself that all articles furnished pursuant to this contract comply in all respects to the specifications, are free from latent or patent defects in design and construction, are suitable and adequate for the purpose intended and guarantees that they will give efficient service for a period of 1 year after acceptance, under the specified conditions; and the Contractor shall, at his own expense, replace, f.o.b. original point of delivery, any parts proving defective during such period. The Contractor further warrants, and upon demand will, at its own expense, defend the title to all materials, articles, or work furnished or done hereunder. The Contractor further warrants that the same are free from any and all claims and demands in respect to the foregoing.

None of the foregoing shall be construed as relieving the Contractor of any warranty implied by law.

ART. 28. *Failure to Meet Requirements.*—Should inspection or test of any piece of apparatus show that it does not meet the guarantees or other requirements of the contract, the Engineer may reject the apparatus or may direct the Contractor to proceed at once to furnish such new parts as may be necessary to bring it up to requirements. All expense of furnishing and delivering new parts or alterations to existing parts shall be borne by the Contractor.

ART. 29. *Right to Operate Apparatus.*—If the apparatus or any part thereof or the operation thereof after installation fails to meet the guarantees or other requirements of the contract, the Authority shall have the right to operate the apparatus until it can be taken out of service without injury to the Authority, for correction of defects, errors, or omissions: *Provided*, That the period of such operation pending the correction of defects, errors, or omissions shall not exceed 1 year without further agreement. Nothing in this paragraph shall be construed as relieving the Contractor of any of the requirements of this contract and specifications.

ART. 30. *Walsh-Healey Act.*—The attached representations and stipulations pursuant to Public Act No. 846, Seventy-fourth Congress, are made a part of this contract.

ART. 31. *Services of Erecting Engineer.*—The Contractor shall, if required, furnish at such times and for such periods as required by the Authority, one or more competent Erecting Engineers, as needed, to supervise, direct, and be responsible for the installation, charging, and putting into operation and to assist the Authority in testing, in the field, of the equipment furnished under this contract. For the services of the Erecting Engineers the Contractor will be paid the amount per day (consisting of 8 hours), or fraction of a day, including Sundays and legal holidays, stated in the Bid Schedule. The payment will cover the entire period of time that the Erecting Engineer is in the service of the Authority, including not more than the time required to travel by the most direct rail route from the Contractor's fabricating plant to the site of erection and return. Railroad and sleeping-car fares and other necessary transportation expenses will be paid by the Authority (in accordance with TVA Travel Regulations) but no payment will be made for subsistence or other personal expense while en route or elsewhere. The Authority will furnish the Contractor a copy of the above-mentioned Travel Regulations on request.

In Witness Whereof, the parties hereto have caused this contract to be executed the day and year first above written.

[SEAL]

Contractor

Attest:

By____________________________

____

TENNESSEE VALLEY AUTHORITY,

By____________________________

TENNESSEE VALLEY AUTHORITY

STATE OF____________________ } ss:
COUNTY OF__________________ }

On this________day of____________________, 193__, before me personally appeared______________________________, to me known to be the person(s) described in and who executed the foregoing instrument and acknowledged that __he__ executed the same as__________free act and deed.

Witness my hand and seal at office this__________day of____________________ 193__.

____
Notary Public.

My commission expires:

____

STATE OF______________________ } ss:
COUNTY OF__________________ }

On this__________day of____________________, 193__, before me personally appeared________________________________, to me personally known, who, being by me duly sworn, did say that he is the______________________________of the corporation named as Contractor in the foregoing instrument and the said instrument was signed in behalf of said corporation by authority of its board of directors and did acknowledge said instrument to be the free act and deed of said corporation.

Witness my hand and seal at office this__________day of____________________, 193__.

Notary Public.

My commission expires:

TVA 1847

Walsh-Healey Act.—The following representations and stipulations pursuant to Public Act No. 846, Seventy-fourth Congress, are made a part of this contract:

(*a*) The Contractor is the manufacturer of or a regular dealer in the material, supplies, articles, or equipment to be manufactured or used in the performance of the contract.

(*b*) All persons employed by the Contractor in the manufacture or furnishing of the materials, supplies, articles, or equipment used in the performance of the contract will be paid, without subsequent deduction or rebate on any account, not less than the minimum wages as determined by the Secretary of Labor to be the prevailing minimum wages for persons employed on similar work or in the particular or similar industries, or groups of industries currently operating in the locality in which the materials, supplies, articles, or equipment are to be manufactured or furnished under the contract: *Provided, However*, That this stipulation with respect to minimum wages shall apply only to purchases or contracts relating to such industries as have been the subject matter of a determination by the Secretary of Labor.

(*c*) No person employed by the Contractor in the manufacture or furnishing of the materials, supplies, articles, or equipment used in the performance of the contract shall be permitted to work in excess of 8 hours in any 1 day or in excess of 40 hours in any 1 week, unless such person is paid such applicable overtime rate as has been set by the Secretary of Labor.

(*d*) No male person under 16 years of age and no female person under 18 years of age and no convict labor will be employed by the Contractor in the manufacture or production or furnishing of any of the materials, supplies, articles, or equipment included in the contract.

(*e*) No part of the contract will be performed nor will any of the materials, supplies, articles, or equipment to be manufactured or furnished under said contract be manufactured or fabricated in any plants, factories, buildings, or surroundings or under working conditions which are unsanitary or hazardous or dangerous to the health and safety of employees engaged in the performance of the contract. Compliance with the safety, sanitary, and factory-inspection laws of the State in which the work or part thereof is to be performed shall be prima facie evidence of compliance with this subsection.

(*f*) Any breach or violation of any of the foregoing representations and stipulations shall render the party responsible therefor liable to the United

States of America for liquidated damages, in addition to damages for any other breach of the contract, in the sum of $10 per day for each male person under 16 years of age or each female person under 18 years of age, or each convict laborer knowingly employed in the performance of the contract, and a sum equal to the amount of any deductions, rebates, refunds, or underpayment of wages due to any employee engaged in the performance of the contract; and, in addition the agency of the United States entering into the contract shall have the right to cancel same and to make open-market purchases or enter into other contracts for the completion of the original contract, charging any additional cost to the original contractor. Any sums of money due to the United States of America by reason of any violation of any of the representations and stipulations of the contract as set forth herein may be withheld from any amounts due on the contract or may be recovered in a suit brought in the name of the United States by the Attorney General thereof. All sums withheld or recovered as deductions, rebates, refunds, or underpayments of wages shall be held in a special deposit account and shall be paid, on order of the Secretary of Labor, directly to the employees who have been paid less than minimum rates of pay as set forth in such contracts and on whose account such sums were withheld or recovered: *Provided*, That no claims by employees for such payments shall be entertained unless made within 1 year from the date of actual notice to the Contractor of the withholding or recovery of such sums by the United States of America.

(*g*) The Contractor shall post a copy of the stipulations in a prominent and readily accessible place at the site of the contract work and shall keep such employment records as are required in the Regulations under the act available for inspection by authorized representatives of the Secretary of Labor.

(*h*) The foregoing stipulations shall be deemed inoperative if this contract is for a definite amount not in excess of $10,000.

The stipulations above enumerated are not applicable in the following instances:

(*a*) Where the contracting officer is authorized by statute or otherwise to purchase in the open market without advertising for proposals;

(*b*) Where the contract relates to perishables, including dairy, livestock, and nursery products ("perishables" cover products subject to decay or spoilage and not products canned, salted, smoked, or otherwise preserved);

(*c*) Where the contract relates to agricultural or farm products processed for first sale by the original producers;

(*d*) Where the contract is by the Secretary of Agriculture for the purchase of agricultural commodities or the products thereof;

(*e*) Where the contract is with a common carrier for carriage of freight or personnel by vessel, airplane, bus, truck, express, or railway line, where published tariff rates are in effect;

(*f*) Where the contract is for the furnishing of service by radio, telephone, telegraph, or cable companies, subject to the Federal Communications Act of 1934.

TVA 1851

TENNESSEE VALLEY AUTHORITY

Schedule of Labor Classifications on Construction Work and Temporary Operating and Maintenance Work and Hourly Rates of Pay

This schedule, effective January 1, 1938, must be posted in a conspicuous place, easily accessible to employees, on all TVA contract work on which, in accordance with the stipulations included in the invitation to bids, the payment of minimum rates of pay equal to the rates listed in this schedule, is required. Copies for posting will be furnished upon request. When higher rates are being paid by the contractor a schedule of rates actually being paid may be substituted for this schedule of required minimum rates of pay.

The division of occupations into classes of work shall give due and adequate recognition to intelligence, skill, training, experience, and responsibility required. The classification of occupations into classes and grades of work need not be bound by traditional rules and customs. In accordance with this schedule a minimum rate for hourly employees engaged in construction occupations listed herein is 47½ cents per hour, below which none of the listed occupations will be classified.

Skilled-labor classifications shall include work requiring considerable training and/or experience in performance of the work, with a minimum of supervision. Skilled work includes that requiring the training and experience of journeymen-mechanics who can perform all the more important operations of their trades without special instruction or detailed supervision. Skilled work involves the use of complex tools and equipment, judgment in the use of materials, and accuracy in performance of the work.

Classifications below the skilled level shall include:

(*a*) Work performed by apprentices, helpers, and tenders to the skilled occupations. Tenders are laborers assisting construction trades and are not expected to use the tools of the skilled trades.

(*b*) Other work calling for a limited degree of skill and experience, sufficient to enable the individual to proceed efficiently with his duties after a short breaking-in period, and such other work as requires only a moderate degree of training, skill, and responsibility. Such work may require an ability to use simple hand tools or operate power-driven tools, machines, or equipment on repetitive operation.

(*c*) Work performed by unclassified or unskilled labor includes operations of a simple routine nature which require little or no formal education or previous training or experience. The use of simple hand tools or equipment may be involved in unskilled work. Unskilled work is usually performed under close supervision. It includes such work as is done by watchmen, cooks' helpers, excavation laborers, etc.

The following labor classifications and rates shall apply to:

Roadside production of material, whether by subcontractor or otherwise.

All hauling of material from roadside quarries and pits, from railroad or water-delivery points, or from local sources of production to the site of the work, whether the work be done by the contractor or by a subcontractor.

Concrete proportioning plants, from which material is used wholly on this contract or on contracts under the supervision of the Authority.

The provisions relating to hours of employment shall not apply to camp help, i. e., cooks, cooks' helpers, hostlers, and stablemen.

The minimum wages specified herein shall be exclusive of any charges for medical examination or insurance. No individual employed on the project in other than an administrative position shall be paid less than the minimum rate for unskilled labor

Copies of all pay rolls for work performed under this contract (whether done by the contractor or under a subcontract, or otherwise), certified under oath, by the contractor or his authorized representative, shall be filed with the engineer showing the name of each employee, the State and county of his bona fide residence, the class of work performed, the hours worked each day, the wage rate paid, the total amount earned and deductions for board, if any.

The pay rolls shall be divided into three sections under which shall be appropriately grouped (1) executives, administrative and supervisory employees; (2) skilled labor, and (3) unskilled labor. Pay rolls shall be submitted for each calendar month (or part thereof) not later than the fifth day of the following month. Deviation from this procedure will not be permitted. The contractor's time books shall be open to the inspection of the engineer at any time.

Where camps are operated by the contractor, or by person affiliated with the contractor, a deduction on the pay roll of more than sixty cents (60¢) per day for board and lodging will be considered a violation of the minimum wage specified herein. No deduction from the wages of any skilled or unskilled laborer shall be made on account of goods purchased or obligations incurred in any commissary or store owned, leased, or otherwise controlled by the contractor. Obligations so incurred shall be subject to collection only in the same manner in which obligations in the ordinary course of business are collectible. Charges in excess of a fair market price for goods purchased from stores owned, leased, or otherwise controlled by the contractor will not be permitted.

No fee of any kind shall be asked or accepted by the contractor or any of his agents from any person who obtains work on the project nor shall any person be required to pay any fee to any other person or agency obtaining employment for him on the project.

No skilled or unskilled labor shall be charged for any tools used in performing their respective duties except for loss or damage thereto.

Every employee on the work covered by this contract shall be permitted to lodge, board, and trade where and with whom he elects, and neither the con-

tractor nor his agents, nor his employees, shall directly or indirectly require as a condition of employment that an employee shall lodge, board, or trade at a particular place or with a particular person.

No charge shall be made for any transportation furnished by the contractor or his agents to any person employed on the work.

No individual shall be employed as a skilled or unskilled laborer on this contract except on a wage basis, but this shall not be construed to prohibit the rental of teams, trucks, or other equipment from individuals. No such rental agreement, or any charge for food, gasoline, supplies, or repairs, on account of such agreement, shall cause any deduction from the wages accruing to any employee.

All of the above provisions shall also apply where work is to be performed by piece work, station work, or by subcontract. The minimum wage shall be exclusive of equipment rental on any equipment which the worker or subcontractor may furnish in connection with his work.

TENNESSEE VALLEY AUTHORITY

Schedule of Labor Classifications and Hourly Rates of Pay on Construction Work and Temporary Operating and Maintenance Work Effective January 1, 1938 (Revised)

A schedule of labor classifications and hourly rates of pay on construction work and temporary operating and maintenance work was included. These hourly rates of pay were adjusted from time to time to meet labor conditions. The rates that were in effect during the various periods of construction of Norris Dam are given in appendix I.

TVA 363 (1-38)

TENNESSEE VALLEY AUTHORITY

Performance Bond

Know All Men by These Presents, That we,________________________________ as principal, and____ as surety, are held and firmly bound unto the Tennessee Valley Authority, hereinafter called the Authority, in the penal sum of____________________________ Dollars ($__________), lawful money of the United States, for the payment of which sum well and truly to be made, we bind ourselves, our heirs, executors, administrators, and successors, jointly and severally, firmly by these presents.

The Condition of This Obligation is Such, That whereas the principal entered into a certain contract, hereto attached, with the Authority, dated___________ ________, 193__, for____

Now, Therefore, if the principal shall well and truly perform and fulfill all the undertakings, covenants, terms, conditions, and agreements of said contract during the original term of said contract and any extension thereof that may be granted by the Authority, with or without notice to the surety, and during the life of any guaranty required under the contract, and shall also well and truly perform and fulfill all the undertakings, covenants, terms, conditions, and agreements of any and all duly authorized modifications of said contract that may hereafter be made, notice of which modifications to the surety being hereby waived, then this obligation to be void; otherwise to remain in full force and virtue.

In Witness Whereof, the above-bounden parties have executed this instrument under their several seals this__________day of__________________, 193__, the name and corporate seal of each corporate party being hereto affixed and these presents duly signed by its undersigned representative, pursuant to authority of its governing body.

[SEAL]

____
Principal.

Address______________________
____

By__________________________
____
(Title)

Attest:

[SEAL]

Surety.
Address--------------------

By-------------------------

(Title)

Attest:

The rate of premium of this bond is $----------per thousand.
The total amount of premium charged, $----------

(The above must be filled in by corporate surety)

TVA 363–A

TENNESSEE VALLEY AUTHORITY

PAYMENT BOND

(Pursuant to Act of Congress, 49 Stat. 1011)

Know All Men by These Presents, That We, ---------------------------------- as principal, and -- as surety, are held and firmly bound unto the Tennessee Valley Authority, hereinafter called the Authority, in the penal sum of ------------------------ -- Dollars ($----------), lawful money of the United States, for the payment of which sum well and truly to be made, we bind ourselves, our heirs, executors, administrators, and successors, jointly and severally, firmly by these presents.

The Condition of This Obligation is Such, That whereas the principal entered into a certain contract, hereto attached, with the Authority, dated---------- 193 , for -- --

Now, Therefore, if the principal shall well and truly make payment to all persons supplying labor and materials in the prosecution of the work provided for in said contract, and any authorized modifications of said contract that may hereafter be made, notice of which modifications to the surety being hereby waived, then this obligation to be void; otherwise to remain in full force and virtue.

In Witness Whereof, the above-bounden parties have executed this instrument under their several seals this----------day of--------------------193 , the name and corporate seal of each corporated party being hereto affixed and these presents duly signed by its undersigned representative, pursuant to authority of its governing body.

Principal.
Address --------------------

By-------------------------

(Title)

[SEAL]

Attest:

Surety.
Address --------------------

By -------------------------

(Title)

[SEAL]

Attest:

The rate of premium on this bond is $----------per thousand.
The total amount of premium charged, $----------

(The above must be filled in by corporate surety)

Specifications

The requirements for each purchase were specified in complete detail in order that there could be no misunderstanding as to exactly what was desired. The specifications for the Norris Dam slide gates, included in this appendix, are typical. Some of the specifications, such as those for the turbines or for the generators were considerably longer, although the majority of specifications were shorter. Major contracts for the Norris project included the following articles or services:[1]

STRUCTURAL STEEL FRAMING AND BRIDGES

Spillway bridge	October 28, 1935
McClintic-Marshall Corporation, Bethlehem, Pa	$25, 674. 00
Steel framing for powerhouse and structures and supports for electrical installation	June 29, 1935
Virginia Bridge & Iron Co., Roanoke, Va	$62, 980. 00

MISCELLANEOUS METAL WORK

Handrail supports and lighting fixtures	June 30, 19[illegible]
General Electric Supply Corporation, Knoxville, Tenn	$9, 403. 22
Miscellaneous aluminum work	July 7, 1936
Newman Bros., Cincinnati, Ohio	$9, 736. 00
Handrailing and structural steel	October 30, 1936
Fabricated Steel Products Co., Wheeling, W. Va	$982. 00
Transformer handling equipment	April 25, 1936
Atlas Car & Foundry Co., Cleveland, Ohio	$5, 845. 00

MECHANICAL AND HYDRAULIC EQUIPMENT

Outlet sluice gates, operating equipment, and conduit liner castings	August 6, 1934
Hardie-Tynes Manufacturing Co., Birmingham, Ala	$151, 447. 92
Outlet trash racks:	
Atchley & White Machine Shop and Foundry, Knoxville, Tenn	November 15, 1934, $2, 471. 12
Knoxville Iron Co., Knoxville, Tenn	November 15, 1934, $9, 596. 30
Drum gates, seats, and hinge anchors	October 28, 1935
Virginia Bridge & Iron Co., Roanoke, Va	$103, 480. 00
Drum gate operating equipment and piping	October 31, 1935
Koppers Construction Co., Fort Wayne, Ind	$3, 774. 00
Elevator	March 4, 1936
Westinghouse Electric Elevator Co., Chicago, Ill	$13, 649.38
Gallery pump	August 8, 1935
Peerless Pump Division, Food Machinery Corporation, Massilon, Ohio	$978. 94
Trash screens and air cleaning system for intake	March 15, 1935
A. J. O'Leary & Sons Co., Chicago, Ill	$19, 974. 44
Intake gates—tractor type	August 23, 1935
Bartlett-Hayward Co., Baltimore, Md	$126, 140. 89
Intake gate guides	June 24, 1935
Union Steel Casting Co., Pittsburgh, Pa	$9, 549. 00
Intake gate metalwork	April 29, 1935
Virginia Bridge & Iron Co., Roanoke, Va	$5, 071. 00

STEEL PENSTOCKS

Two 20-foot diameter penstocks	June 15, 1934
Chicago Bridge & Iron Co., Chicago, Ill	$103, 439. 26

[1] Figures shown are gross contract costs and do not include liquidated damage payments which were made in several instances.

POWERHOUSE SUPERSTRUCTURE

Air conditioning and ventilating	March 12, 1936
York Ice Machinery Co., York, Pa	$14,000.00
Double door for east wall	April 18, 1936
John Harsch Bronze Co., Cleveland, Ohio	$4,162.00
Aluminum sash	May 1, 1936
The Kawneer Co., Niles, Mich	$12,300.00
Acoustical tile	May 21, 1936
Acoustic Engineering Co., Atlanta, Ga	$271.00
Furring, lathing, and plastering	May 25, 1936
Hopton Bros., Nashville, Tenn	$4,926.12
Furring, lathing, and plastering	September 4, 1936
Hopton Bros., Nashville, Tenn	$1,413.00
Terrazzo	May 27, 1936
Art Mosaic & Tile Co., Toledo, Ohio	$2,146.75
Glass and glazine	June 29, 1936
Standard Glass Co., Knoxville, Tenn	$1,454.64
Metal and aluminum doors	June 30, 1936
John Harsch Bronze Co., Cleveland, Ohio	$16,172.75
Linoleum	July 13, 1936
Fowler Brothers Co., Knoxville, Tenn	$528.03
Sheet metal ducts	July 6, 1936
J. W. Brooks & Sons, Chattanooga, Tenn	$1,257.00
Quarry tile	July 13, 1936
Atlanta Tile & Marble Co., Atlanta, Ga	$6,747.00
Opaque structural glass	August 20, 1936
Standard Glass Co., Knoxville, Tenn	$5,541.30
Precast roof slabs	March 26, 1935, May 21, 1936
Southern Cast Stone Co., Knoxville, Tenn	$9,696.00
Lighting fixtures:	
Day-Brite Reflector Co., St. Louis, Mo	June 27, 1936, $3,440.50
Graybar Electric Co., Knoxville, Tenn	June 27, 1936, $38.43
Kurt Versen, Inc., New York, N. Y	June 22, 1936, $193.32
Graybar Electric Co., Knoxville, Tenn	June 22, 1936, $266.87
Plumbing fixtures	January 22, 1936
Crane Co., Knoxville, Tenn	$1,554.34

TURBINES AND GOVERNORS

Two 66,000-horsepower Francis type hydraulic turbines	October 22, 1934
Newport News Shipbuilding & Dry Dock Co., Virginia	$467,930.00
Turbine governors	August 23, 1935
Woodward Governor Co., Rockford, Ill	$27,510.00
Two water flow recorders and indicators	August 23, 1935
Bailey Meter Co., Cleveland, Ohio	$1,615.00

ELECTRICAL EQUIPMENT

Generators (complete with auxiliary equipment)	January 17, 1935
Westinghouse Electric & Manufacturing Co., Knoxville, Tenn	$880,120.00
Transformers	April 26, 1936
General Electric Co., Knoxville, Tenn	[2] $6,573.00
Auxiliary switchboards	February 13, 1936
I. T. E. Circuit Breaker Co., Philadelphia, Pa	$26,864.40
Main control switchboards	April 1, 1936
General Electric Co., Knoxville, Tenn	[2] $54,652.00
Annunciator system and one automatic type oscillograph (complete with all accessories)	April 1, 1936
Westinghouse Electric & Manufacturing Co., Knoxville, Tenn	$2,225.00
Neutral oil circuit breaker, reactor and surge protective equipment	April 27, 1936
Westinghouse Electric & Manufacturing Co., Knoxville, Tenn	$12,044.00
Disconnecting switches	April 27, 1936
Southern States Equipment Corporation, Birmingham, Ala	$1,579.50
Fire extinguishing equipment (for generators, oil purifier room, and oil storage room)	February 3, 1936
Walter Kidde & Co., New York, N. Y	$8,986.00

ELECTRICAL EQUIPMENT—continued

Control battery and charging sets	December 12, 1935
The Electric Products Co., Cleveland, Ohio	[2] $2, 630. 00
Frequency source and frequency and load controls	May 26, 1936
Leeds & Northrup Co., Philadelphia, Pa	[2] $11, 516. 93
Carrier current equipment	May 11, 1936
General Electric Co., Knoxville, Tenn	[2] $21, 611. 00
Oil purifier	May 20, 1935
De Laval Separator Co., New York, N. Y	[2] $5, 872. 30
Two metal bus housings	March 20, 1936
Frank R. Adams Electric Co., St. Louis, Mo	$4, 138. 00
Telephone cable	July 8, 1936
General Cable Corporation, Philadelphia, Pa	[2] $3, 094. 29

[2] Amount shown is for Norris project only. Similar material was purchased at the same time for other projects on the same contract.

SWITCHYARD

Structural steel towers, trusses, and framework for mounting and supporting the switchyard equipment	April 1, 1935
Bethlehem Steel Co., Bethlehem, Pa	$11, 778. 95
Wire and cable	June 29, 1936
Anaconda Wire & Cable Co., New York, N. Y	$321. 10
Racks for cable supports	May 22, 1936
Lehigh Structural Steel Co., New York, N. Y	$1, 270. 00
Main step-up power transformers and neutral grounding reactor	August 29,1935
General Electric Co., Knoxville, Tenn	$282, 849. 00
161-kv. oil circuit breakers	September 12, 1935
General Electric Co., Knoxville, Tenn	$172, 156. 00
Lightning arresters	April 16, 1936
General Electric Co., Knoxville, Tenn	$8, 418. 00
Chain link fence and accessories	June 15, 1936
Wimberly & Thomas Hardware Co., Birmingham, Ala	$966. 50

STATION MECHANICAL AND OPERATING EQUIPMENT

250-ton double trolley, motor-operated traveling crane	May 31, 1935
Harnischfeger Sales Corporation, Milwaukee, Wis	$61, 180. 00
Air compressors:	
Worthington Pump & Machine Co., Harrison, N. Y	June 17, 1935, $1, 408. 00
Sullivan Machinery Company, Chicago, Ill	June 17, 1935, $3, 071. 00
Air receivers	June 17, 1935
Schramm, Inc., West Chester, Pa	$597. 00
Complete water distillation apparatus	June 18, 1936
McKesson-Doster-Northington, Birmingham, Ala	$235. 75
Water level gages	September 28, 1936
Julien P. Friez & Sons, Baltimore, Md	$926. 66
Two oil-pumping units:	
George D. Roper Corporation, Rockford, Ill	March 10, 1936, $151. 35
Schutte & Koerting Co., Philadelphia, Pa	March 9, 1936, $507. 50
Complete telephone and signalling systems	March 23, 1936
American Automatic Electric Sales Co., Chicago, Ill	$8, 523. 24

SPECIFICATIONS FOR SLIDE GATES

CONTENTS

SPECIFICATIONS FOR SLIDE GATES—Continued

CONTENTS—Continued

SPECIFICATIONS FOR SLIDE GATES FOR NORRIS DAM

1. *The requirement.*—It is required that there be furnished and delivered, complete, in accordance with these specifications and the drawings, listed in paragraph 3 hereof, eight sets of 5-foot 8-inch by 10-foot high-pressure slide gates, together with frames, hoists, automatic and semiautomatic gate hangers, conduit linings, and all other appurtenant parts, for installation in the outlet works at Norris Dam. All materials will be installed by the Authority. The air vent, by-pass, oil pump and control piping is not covered by these specifications.

2. *Description of emergency and regulating gates.*—The gates will be installed in eight sets as shown on the drawings. Each set of gates will consist of two slide gates installed in tandem, with four sections of conduit lining above the upstream gate and four sections of lining below the downstream gate. The downstream or service gates will be used for regulating the discharge of water through the dam, and will be equipped with automatic gate hangers. The upstream gates will be only for emergency use in case of damage to the sluice gates or to permit necessary maintenance and repairs, and will be equipped with semiautomatic gate hangers. The gates will be operated under a maximum head of 169 feet, but will be subjected to a hydrostatic head of 182 feet. Operation will be by means of hydraulic cylinders using oil as a medium, with a working pressure of 1,000 pounds per square inch. All air-vents, by-pass, oil pump and control piping will be furnished by the Authority, and is not covered by these specifications. The automatic gate hangers will be attached to the heads of the hydraulic cylinders of the service gates, and are for the purpose of holding the gates in any open position. Each automatic gate hanger will be hydraulically operated, using oil as a medium, by a valve connected to the cylinder hoist pressure line which will admit oil under pressure to a small piston acting against springs to clamp the jaws of the hanger to the projecting stem of the gate. When closing the gate the pressure is released on the hanger piston. The springs then react to release the pressure on the jaws on the gate stem. Mercury switches, to be furnished by the Authority will be installed to indicate to the operator whether the jaws are open or closed. The semiauto-

matic gate hangers will be suspended above the hydraulic cylinders of the emergency gates and are for the purpose of holding the gates in the open position. When an emergency gate is open a cap on an extension of the gate stem will be engaged by the hanger hooks and before closing the gate the hooks must be released by manual operation. A safety stud is provided which is so proportioned that it will break should power be applied to close the gate without first releasing the hanger.

3. *Drawings.*—The following drawings are a part of these specifications:

5 feet 8 inches by 10 feet 0 inches Slide Gates

1. 232–D–232 Tandem installation (sheet 1 of 8).
2. 232–D–233 Assembly (sheet 2 of 8).
3. 232–D–234 Upstream frame (sheet 3 of 8).
4. 232–D–235 Downstream frame (sheet 4 of 8).
5. 232–D–236 Bonnet cover (sheet 5 of 8).
6. 232–D–237 Leaf and seats (sheet 6 of 8).
7. 232–D–238 24-inch hydraulic hoist (sheet 7 of 8).
8. 232–D–239 Conduit lining, bolts and list of parts (sheet 8 of 8).
9. 232–D–240 Semiautomatic gate hanger—20,000 pounds capacity.

Hydraulic Automatic Gate Hanger No. 2

10. 232–D–246 Assembly (sheet 1 of 6).
11. 232–D–247 Cylinder (sheet 2 of 6).
12. 232–D–248 Cylinderhead, clamp jaw, runner (sheet 3 of 6).
13. 232–D–249 Base, piston valve, hinge pin (sheet 4 of 6).
14. 232–D–250 Bolts, studs, list of parts 1 to 39 (sheet 5 of 6).
15. 232–D–251 Miscellaneous and list of parts 42 to 90 (sheet 6 of 6).

The contractor will not be held responsible for the correctness or sufficiency of designs, but he shall carefully check the drawings and advise the Authority of any errors or omissions discovered. The contractor shall prepare, without charge to the Authority, all necessary shop drawings covering the materials to be furnished under these specifications, and shall be responsible for the correct fitting of all parts. Unless otherwise specifically provided in the schedule and specifications, or on the drawings, the contractor shall furnish all of the materials, accessories, and appurtenant parts called for in the specifications or shown on the drawings.

Anything shown on the drawings and not mentioned in the specifications, or called for in the specifications and not shown on the drawings, shall be furnished the same as if called for or shown in or on both. The contractor will be furnished such additional copies of the specifications and drawings as may be required for carrying out the work. One contact print and one Van Dyke negative of the original drawings from which the accompanying reductions were made will be furnished to the contractor for construction purposes.

4. *Workmanship and defective work and materials.*—All work shall be done and completed in a thorough workmanlike manner, notwithstanding any omissions from these specifications or drawings. Tolerances and clearances specified on the drawings shall be closely adhered to, and the machine work shall in all cases be of high-grade workmanship and finish, carefully performed to the satisfaction of the Authority. Where tolerances are not specified on the drawings, the contractor shall follow the best modern shop practice for apparatus of the type covered by these specifications and drawings, due consideration being given to the special nature of functions of the parts and to the corresponding accuracy required to secure proper operation. The contractor shall guarantee all materials and workmanship furnished by him to be free from injurious defects. He shall replace, free of cost to the Authority, any defective materials or workmanship discovered during erection, and shall pay the actual cost to the Authority of the correction in the field of any errors for which he is responsible.

5. *Inspection and tests.*—All material furnished and all work done shall be subject to rigid inspection and no material shall be shipped until all tests, analyses, and final inspection have been made or certified copies of reports of tests, and analyses, or manufacturer's guarantee shall have been accepted. Test

specimens and samples for analyses shall be properly boxed and prepared for shipment if required. Unless otherwise specifically provided herein, all metals covered by these specifications shall be furnished in accordance with the requirements of Federal specification QQ–M–151, General Specifications for Inspection of Metals, which specification covers certain requirements which are common to all detail specifications for metal and provides means for determining whether the technical requirements of the detail specifications are being met. Specimens for physical test of gate stems and shafting shall be taken from midway between the center and outside of a full-size elongation of stem or shaft after rolling or forging and heat-treating.

6. *Patterns.*—The prices bid in the schedules shall include the cost of all necessary patterns. Ample fillets shall be used to avoid sharp corners or abrupt changes in cross section. Patterns will remain the property of the contractor. All dimensions shown on the drawings are net and the required draft shall be added.

7. *Machine finish.*—Where finished surfaces are specified on the drawings, the type of finish where not otherwise specified must be that most suitable for the part to which it applies and shall be smooth, semismooth, or rough as defined herein. Where a smooth finish is specified or required, the machine work shall be performed in such a manner as to produce smooth surfaces practically free from tool and chatter marks. Pronounced tool marks or other defects on such surfaces will be cause for rejection of the part. Where semismooth finish is specified or required, smooth surfaces shall be produced but small tool marks will be allowed. Where rough finish is specified or required, rough machining sufficient only to produce a plane surface true to dimensions will be allowed. In general, a smooth finish will be required for all surfaces in sliding contact; a semismooth or commercial finish for surfaces to be in permanent contact where a tight joint is required, and a rough finish for all other machined surfaces.

8. *Slide gates.*—Gate frames and gates shall be so accurately machined that the sliding surfaces will be in the same plane and the gates will bear uniformly on the frames. The interior surfaces at the flanged joints between gate frames and between gate frames and conduit lining sections shall match within one-quarter (¼) inch at any point, and all off-sets shall be chipped or ground to a bevel of one in three to provide a reasonably smooth flow line. Gate sills shall be filled with babbitt which shall be thoroughly hammered after pouring and then machined to a uniform surface. Hoist stems shall be straight and round and of uniform section and the sliding surfaces shall be smooth and polished. Hoist cylinders may be constructed of seamless forged steel or of welded plate steel and butt welded to forged steel flanges. The cylinders shall be bored to true circles and to the tolerances shown, and the inside shall have a smooth finish, and if necessary shall be ground. If made in more than one section the inside surfaces of adjacent sections shall be continuous. The outside shall be rough machined to present a neat appearance. Flange faces shall be finished at right angles to the bore.

9. *Shop inspection and test.*—(*a*) *Slide gates, conduit linings, and hoists.*—Each set of slide gates including gates, frames, bonnets, and conduit linings shall be completely assembled in the shop for inspection and to insure that all parts fit accurately and are in proper alignment. Each gate shall be opened and closed several times until the gate functions properly to the satisfaction of the Authority. Each hoist cylinder shall be subject to a test pressure of 1,000 pounds per square inch on each side of the piston, using lubricating oil having a viscosity at 100° F. of between 200 and 300 Saybolt and under this pressure there shall be no more than a trace of leakage past the piston. The finished surface of bolted joints in sections that are to be shipped assembled, and the shanks of the bolts used, shall be smoothly coated with a thin mixture of white lead in oil and graphite or similar material before being assembled.

(*b*) *Automatic gate hangers.*—Each automatic gate hanger shall be assembled in the shop for inspection and test. Springs shall be subjected to load tests specified on the drawings and sorted into pairs of as near the same capacities as possible. Each gate hanger shall be operated several times by means of oil pumped into its operating cylinders until the hanger operates properly to the satisfaction of the Authority. The operating cylinders shall then be subjected to a test pressure of 1,000 pounds per square inch on the side of the pistons next to the hanger, using oil as specified for testing the hoist cylinders, and under this pressure there shall be not more than a trace of leakage past

either the operating cylinder or the valve pistons. Test shall also be made using a pump having a capacity of not less than 30 gallons per minute to insure that all passages are clear. The contractor shall provide rolled bronze rods of the diameter of the projecting valve stems for insertion in the jaws of the hanger and connection to the piston of a test cylinder which will be provided by the Authority. He shall then, by means of the test cylinder, determine the maximum holding capacity of each hanger. After this test is completed to the satisfaction of the Authority the hanger shall be adjusted to a holding capacity of 20,000 pounds and the spring setting for this load recorded. After all inspection and tests are completed the jack screws shall be backed off until the springs are free and the hanger then boxed for shipment.

(c) *Semiautomatic hangers.*—Each semiautomatic gate hanger shall be assembled in the shop for inspection and test and shall be operated a sufficient number of times to insure that all parts work freely to the satisfaction of the Authority.

10. *Marking.*—All parts of each gate, gate frame, gate hoist, and conduit lining shall be marked and match-marked for identification and to facilitate field assembly, and in addition the gates, frames, hoists, and conduit lining for each of the eight sets of slide gates shall be marked to show the set of which it is a part and the relative position of each part in the set, and a diagram of such marking shall be forwarded by the contractor to the Authority. Where separate shipments are to be made of parts which are to be joined together in the field, the joint between the parts shall be fitted in the shop or a template shall be prepared and each part shall be marked and match-marked before the parts are shipped.

11. *Shop painting.*—After inspection has been completed and the materials accepted for shipment, all surfaces to be painted shall be thoroughly cleaned of all scale, rust, dirt, and grease and shall be painted as follows:

The unfinished surfaces of the slide gates, frames, and conduit linings below the gallery floor line, not to be in contact with concrete, and the under sides of the bonnet covers shall be given one shop coat of bituminous primer or of similar priming coating satisfactory to the Authority. The outside of the bonnet covers, the outside of the hoist cylinders, and the unmachined surfaces of the gate hangers shall be given one shop coat of suitable priming paint and one coat of high-grade black machinery paint. The inside of hoist and hanger cylinders shall be oiled and all openings closed for shipment. All finished surfaces to be exposed to the atmosphere during shipment shall be coated with a heavy rust-preventive compound.

12. *Weighing.*—The contractor shall, in the presence of the inspector, weigh all completed apparatus on the most accurate scales available and a complete list of such net weights, exclusive of boxes, crates, or skids shall be furnished to the Authority. The net finished weight of each of the larger pieces shall also be painted on the piece, or stated on a tag attached thereto, before shipment.

WELDING

13. *Preparation for welding.*—Plates to be joined by welding shall be accurately cut to size and rolled by pressure to the proper curvature, which shall be continuous from the edges. Flattening in the curvature along the edges with correction by blows will not be allowed. The dimensions and shape of the edges to be joined shall be such as to allow thorough fusion and complete penetration, and plates shall be planed if necessary to accomplish this result. The surface of the plates for a distance of one-half inch back from the welding edge shall be thoroughly cleaned of all rust, grease, and scale, to bright metal.

14. *Welding hoist cylinders.*—All welding shall be done in the shop and the welding of both longitudinal and girth joints shall conform to the current "Rules for the Fusion Process of Welding," class 1, of the A. S. M. E. Boiler Construction Code, Unfired Pressure Vessel Section. All joints shall be single "V" butt-welded on automatic electric welding machines by a process that will exclude the atmosphere from the metal of the weld while it is in a molten state. The joints shall be thus welded in such a manner as to insure uniform distribution of the load throughout the welded section with no tendency to produce eccentric loading or shear in the weld or the metal adjacent thereto. The welding process and speed shall be under control at all times and there shall be no greater evidence of oxidation in the metal of the weld than in the metal of the original unwelded plate. If welding is stopped for any reason, special care shall be taken when welding is resumed to get full penetration to the bottom of the

joint and thorough fusion between the weld metal and the plate to weld metal previously deposited. The finish of the welded joint shall be reasonably smooth and free from grooves, depressions, and other irregularities, and there shall be no valley at the center of the weld. If a cylinder shows irregularities after welding, it shall be rerolled to render it practically cylindrical. All welds shall be thoroughly peened. Welded cylinder shells shall be made of plates of sufficient thickness to allow machining inside and outside to produce a smooth finish on the inside and a rough finish on the outside. The steel used in the cylinders shall be of a composition that will produce the best quality of welded joints, with physical properties in conformance with the "Standard Specifications for Boiler and Firebox Steel for Stationary Service" of the American Society for Testing Materials, for "Flange" quality steel.

15. *Test of welded joints.*—Tests of all welded joints shall be made in accordance with the latest revised addenda to the A. S. M. E. Boiler Construction Code, Unfired Pressure Vessel Section, paragraph No. U–68, for class 1 vessels. Every portion of each welded joint shall be radiographed by a powerful X-ray apparatus under a technique that will determine quantitatively the size of a defect with a thickness greater than 2 percent of the thickness of the plate. All X-ray films shall be submitted to the Authority. Any defects repaired after the X-ray examination shall again be X-rayed. Hydrostatic tests of each pipe section at a test pressure of 1,500 pounds per square inch, combined with hammer tests, in accordance with paragraph No. U–77 of the above "Code" will be accepted in lieu of X-ray tests. For determination of the physical properties of welds of longitudinal joints only tension and bend tests in accordance with the above "Code" will be required, from full section test coupons attached to every section. The tensile strength of the joint specimen shall not be less than the minimum of the specified range of the plate used.

MATERIALS

16. *Federal specifications.*—Copies of the Federal specifications referred to herein may be procured from the Superintendent of Documents, Government Printing Office, Washington, D. C., at a price of 5 cents per copy, except specification FF–B–571, the price of which is 10 cents per copy.

17. *Semisteel.*—Semisteel castings shall be made by the cupola process and shall conform to Federal Specification QQ–I–656 for high test gray iron castings (semisteel). Tension test will not be required.

18. *Cast steel.*—Steel castings shall be of medium grade and shall conform to Federal Specification QQ–S–681 for steel castings.

19. *S. A. E. 2,340 steel.*—S. A. E. 2,340 steel shall be a nickel steel conforming to the specifications No. 2340 of the Society of Automotive Engineers. The parts shall be heat treated to give physical properties not less than the following:

Ultimate tensile strength	100,000 pounds per square inch.
Elastic limit	80,000 pounds per square inch.
Elongation in 2 inches	20 percent.
Reduction of area	60 percent.

Chromium plating of safety stud is not required.

20. *Forged steel.*—Forged steel shall conform to the standard specifications of the American Society for Testing Materials for "Carbon Steel and Alloy Steel Forgings," Class C. Serial designation A–18–30. Forgings shall be annealed before machining.

21. *Steel.*—Where "steel" only is specified the contractor may use first-class grade of commercial carbon steel best suited for the purpose for which the part is to be used.

22. *Bolts, studs, and nuts.*—Bolts, studs, and nuts shall conform in all respects to Federal Specification FF–B–571, for bolts, nuts, studs, and tap rivets (and material for same), except that threads made by rolling will not be allowed. Unless otherwise specified, threads shall be free fit.

23. *Gate stems.*—Gate stems shall be made from Monel metal or stainless steel and shall be rolled or forged and heat treated if necessary to produce physical properties not less than the following:

Ultimate tensile strength	90,000 pounds per square inch.
Yield point	70,000 pounds per square inch.
Elongation in 2 inches	25 percent.
Reduction of area	40 percent.

Stainless steel shall contain 17 to 20 percent chromium and 7 to 10 percent nickel.

24. *Cast bronze.*—Bronze castings shall conform in all respects to composition No. 6, Federal Specification QQ–B–691 for bronze castings.

25. *Bronze.*—Where "bronze" only is specified on the drawings, phosphor bronze or gun bronze may be used, providing the material fulfills the requirements of paragraph 24, "Cast bronze."

26. *Rolled bronze.*—The physical properties of rolled bronze shall be not less than the following:

Ultimate tensile strength	60,000 pounds per square inch.
Yield point	30,000 pounds per square inch.
Elongation in 2 inches	30 percent.

27. *Cast manganese bronze.*—Manganese bronze castings shall conform in all respects to Federal Specification QQ–B–726 for manganese bronze castings.

28. *Class "C" bronze.*—The chemical composition of Class "C" bronze shall be as follows:

	Percent
Copper	82.00–83.00
Tin	6.75– 7.50
Lead	4.50– 5.00
Zinc	5.00– 6.00

29. *Class "D" bronze.*—The chemical composition of class "D" bronze shall be as follows:

	Percent
Copper	82.00–83.00
Tin	4.75– 5.50
Lead	7.75– 8.25
Zinc	4.00– 5.00

30. *Babbitt.*—Babbitt for gate sills shall conform to Federal Specification QQ–M–161 for antifriction metal, castings and ingots, grade 3.

31. *Miscellaneous materials.*—Where materials are specified on the drawings, but are not specifically covered herein by detail specifications, the contractor shall furnish high-class commercial grades of materials or articles that are satisfactory to the Authority.

32. *Manufacturer's name plate.*—Cast lettering will not be permitted on any of the castings. The contractor may, however, attach a small name plate, giving manufacturer's name, address, date, etc., on one of the principal castings of each unit.

TVA 1551 (12–37)

Requisition No.

Date

TENNESSEE VALLEY AUTHORITY

ADDENDUM

Bid opening

NOTICE TO BIDDERS:

THIS ADDENDUM IS HEREBY MADE A PART OF THIS INVITATION AND ONE COPY MUST BE SIGNED AND RETURNED WITH EACH COPY OF YOUR BID.

TENNESSEE VALLEY AUTHORITY,
C. H. GARITY, *Director of Purchases.*

(Bidder)

By

(Member of firm or person authorized to sign bid)

Date

Appendix H

ALLOCATION OF COSTS

REPORT OF THE COMMITTEE OF FINANCIAL POLICY OF THE TENNESSEE VALLEY AUTHORITY [1]

To the Board of Directors, Tennessee Valley Authority:

This report on the allocation of the cost of the Wilson, Norris, and Wheeler Dams to the functions served by these multipurpose projects has been prepared pursuant to the authorization by the Board on July 12, 1937, creating a Committee on Financial Policy. Attached to this report are supplementary Notes on Allocation which bring out in detail various considerations underlying and leading to the recommendations that follow.

In the Authority's three completed projects, the investment subject to allocation is $30,120,009 for Wilson Dam, $31,532,120 for Norris Dam, and $32,473,542 for Wheeler Dam—a total of $94,125,671. These figures represent the cost of construction to the Authority of the Wheeler and Norris projects, and the "present value" of the Wilson project as heretofore determined by the Board and approved by the President of the United States in accordance with the provisions of section 14 of the Tennessee Valley Authority Act. Of the total, $4,075,988 is invested in facilities installed wholly for navigation purposes, $2,600,000 in storage used only for flood control, and $23,967,177 in facilities constructed for the generation of electricity. The balance of $63,482,506, 67.5 percent of the total, is the joint investment in the three projects. The committee has been charged with the task of recommending a method by which this joint investment may be divided as between the three functions of flood control, navigation, and power.

The Tennessee Valley Authority Act contemplates that all the dams in the system are to be constructed primarily for navigation and flood control. In section 9 (*a*) of the statute, the Congress authorized the installation of power facilities at the dams and adopted the policy of applying the sales of electric energy as far as practicable to assist in the liquidation or maintenance of the Authority's projects, and in section 14 required an allocation of project costs to the various functions which they serve. As these provisions of the statute are interpreted by the committee, the joint investment of $63,482,506 is to be regarded as a common plant investment, and a procedure must be adopted by which the Board may determine the portion which may reasonably be recovered through the sales of electric energy.

A number of theories of cost allocation were studied carefully by the committee in its attempt to reach a conclusion as to the shares of the joint investment that should be assigned to the various functions. All reasonable possibilities were explored in order to reach a result. A discussion of these theories and the limitations attaching to each appears in the notes supplementing this report. Every method of allocating the common-plant investment necessarily involves assumptions and estimates the formulation of which is dependent on widely varying opinions of individuals.

Of the total investment in the Authority's multipurpose projects the only definite portion that can be associated with any one purpose is the added cost made necessary by the inclusion of that purpose. Whether the required additional expenditure is warranted is a question of policy necessitating the consideration of many factors, the relative importance of which cannot always be determined by a common unit of measurement. The problem is one of judgment rather than scientific calculation.

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[1] Supplementary notes on this allocation accompanied this report and have been printed in H. Doc. No. 709 of the 75th Cong., 3d sess.

This question becomes of considerable importance where a dam-construction project is justified, as required by the Tennessee Valley Authority Act, primarily for navigation and flood-control purposes. Latent water power, an inevitable consequence of the expenditure for navigation and flood control, may be allowed to go to waste, or an additional expenditure may be made to convert it into electrical energy. Power may thus be considered self-supporting when the power revenues are just sufficient to cover the additional costs incident to the establishment and operation of the power facilities. How much higher such revenues should be in order that a portion of the remaining costs may be liquidated is a policy which the act leaves to the Board. The committee's conclusions, are, therefore, in the form of a recommended policy based on judgment and not on any one allocation theory.

Having, then, given due consideration to the requirements of the act and to the various theories of allocation, and taking into account the benefits expected from the navigation and flood-control facilities, as set out in detail in the accompanying supplemental notes, the committee recommends that the combined joint costs of the three-dam system of Wilson, Norris, and Wheeler, amounting to $63,482,506, be allocated to the five functions mentioned in section 14 of the Tennessee Valley Authority Act upon the following percentage basis: Flood control, 25 percent; navigation, 35 percent; power, 40 percent; national defense, none; fertilizer, none.

The three completed projects represent a great national-defense asset, but because they are not in operation for that purpose during peacetime, it appears to be impracticable to attempt to allocate any portion of the investment to their wartime use, and the committee recommends that no part of the investment be assigned to that function. It is further recommended that no part of the joint investment be allocated to fertilizer, since fertilizer operations are being currently charged for services rendered by other departments of the Authority.

Upon this basis the allocation of the total cost of the system would be as follows:

Common expenditures

Purpose	Single-purpose expenditures	Allocation		Total	
		Percent	Amount	Amount	Percent
Flood control	$2,600,000	25	$15,870,627	$18,470,627	20
Navigation	4,075,988	35	22,218,877	26,294,865	28
Power	23,967,177	40	25,393,002	49,360,179	52
National defense					
Fertilizer					
Total	30,643,165	100	63,482,506	94,125,671	100

In the allocation to power recommended above, the normal power capacity of these three projects considered as an integrated system has been assumed. At present less than three-fourths of the estimated capacity required to realize the potential primary and secondary power of the three projects has been installed and is available for service. Current power operations should not be expected to carry fully the amount set forth above as apportionable to power, but for the purposes of simplifying the Authority's accounting the committee does not consider a reduction of the allocations to power to be desirable, pending the time that the maximum capacity is installed.

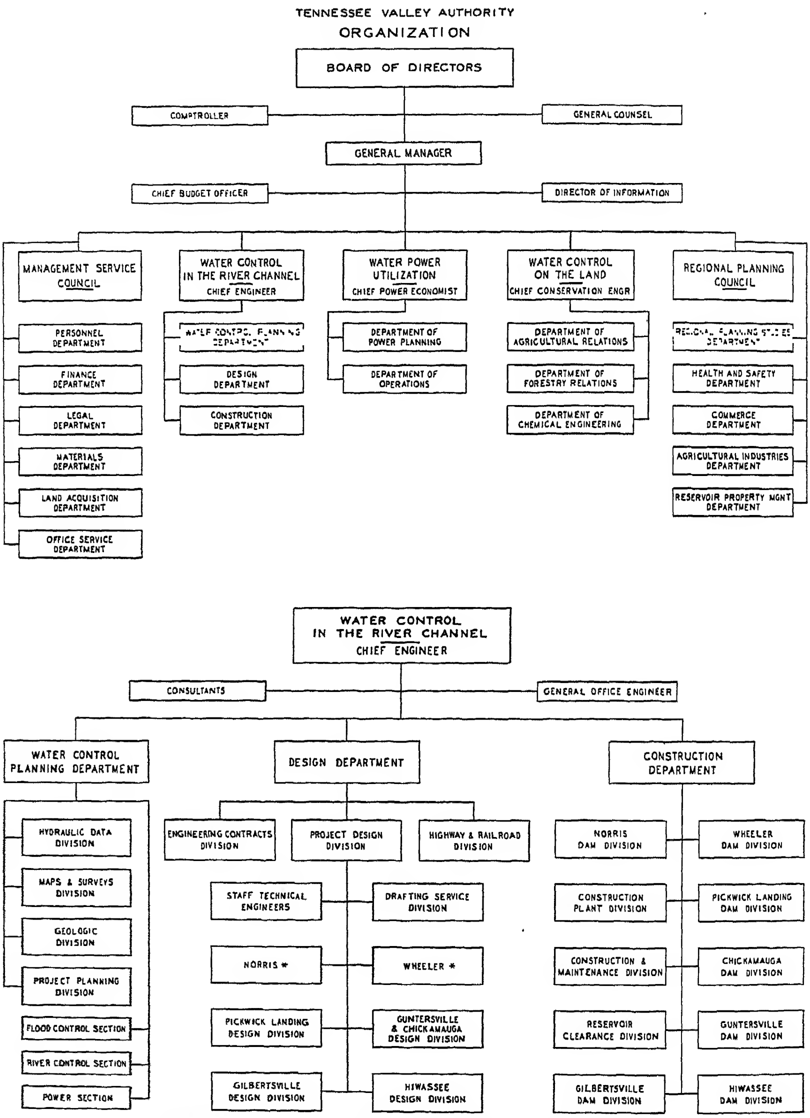
The sole consideration in the preparation of this report has been to recommend an allocation for the purposes of the Authority's accounting as required by sections 9 (*a*) and 14 of the Tennessee Valley Authority Act. These recommendations apply only to the three completed dams as a unit. No allocation of the costs of the various functions to individual dams has been made. The dams are interrelated in their operation to provide a unified result, and any significant allocation to the several uses can be made only from the standpoint of the system rather than by separate projects. Allocation might be deferred until the full development has been reached, but if allocations are henceforth to be made as each construction project is completed, they should continue to be premised on the basis that both main-river and storage dams are to operate as an integrated system.

Acknowledgment is made of the contributions to the committee's discussions by Martin G. Glaeser who served for some months as chairman of the committee; James C. Bonbright and Edward W. Morehouse, who were retained by the Authority as consultants.

Respectfully submitted.

TENNESSEE VALLEY AUTHORITY.
Financial Policy Committee,
E. L. KOHLER,
Comptroller, Chairman.
T. B. PARKER, *Chief Engineer.*
S. M. WOODWARD,
Chief Water-Control Planning Engineer.
W. C. FITTS, *Solicitor.*
J. A. KRUG,
Chief Power Planning Engineer.
PAUL W. AGER,
Chief Budget Officer, Secretary.

JUNE 6, 1938.



* DESIGNED BY THE U. S. BUREAU OF RECLAMATION WITH SOME PARTICIPATION BY THE T. V. A. STAFF

FIGURE 375.—*Organization chart.*

Appendix I

PERSONNEL DATA

Organization.

The chart shown in figure 375 represents the organization of the Authority and particularly the departments for Water Control in the River Channel at the close of the project. Several organizational changes occurred during the construction period and this chart shows only the final arrangement effected. Resolutions of the Board of Directors giving the Authority's organization structure in detail are shown on pages 334 to 365 of the Annual Report of the Authority for the fiscal year ended June 30, 1937.

Personnel.

Barton M. Jones, construction engineer, and Ross White, construction superintendent, were in direct charge of the engineering and construction connected with the dam, powerhouse, and related features. The other principal persons contributing to the Norris project are listed on pages 798 to 802. This list has been limited to those whose responsibility placed them in policy-making positions, insofar as those positions affected the Norris project. Included in the list are a number of employees not directly connected with the actual construction work and who did not devote their efforts entirely to the Norris project but who served in like capacities on several construction projects. It is regretted that space does not permit the listing of all persons who were identified with the project.

Wage scale.

The hourly wage scale was adjusted from time to time so that it might be kept consistent with wages existing in the Tennessee Valley region. The different rate schedules in effect during the construction period are shown on pages 802 and 803.

Employee relationship policy.

The policy governing the relationship between the Authority and its employees was adopted by the Board of Directors as a result of long study and following a series of discussions in which both the employees and management participated. This is given on page 804.

MAJOR SUPERVISORY PERSONNEL

BOARD OF DIRECTORS[1]

Arthur E. Morgan, chairman and chief engineer (until June 17, 1937).

Harcourt A. Morgan, vice chairman.

David E. Lilienthal, director.

John B. Blandford, Jr., coordinator (until May 1936); general manager (after May 1936).

ENGINEERING AND CONSTRUCTION

Administration

Carl A. Bock, assistant chief engineer.

[1] James P. Pope succeeded Arthur E. Morgan as Director on January 27, 1939. Harcourt A. Morgan was made Chairman of the Board March 23, 1938, and David E. Lillienthal was made Vice Chairman January 27, 1939.

ENGINEERING PLANNING

Sherman M. Woodward.

Geology

Edwin C. Eckel.
Berlen C. Moneymaker.
James S. Cullison.
Robert A. Laurence.
William F. Prouty.

Hydraulic Data

Albert S. Fry.
Gene N. Burrell.
Van Court M. Hare.
Benham E. Morriss.
Jackson H. Wilkinson.

Hydraulic Studies

James S. Bowman.
Clifton T. Barker.
Roland A. Kampmeier.
Joseph H. Kimball.
Arthur Schweier.
William L. Voorduin.
Gabriel O. Wessenauer.
Dana M. Wood.

Engineering Services

Ned H. Sayford.
Harry Wiersema, assistant.

reservoir surveys

George D. Whitmore.
John F. Barksdale, assistant.
Roscoe W. Anderson.
Dewitt C. Bishop.
John D. Blagg.
Guy Crawford.
Charles P. Gray.
Gomer D. Hoskins.
William H. Keen.
Samuel H. McBeth.
Clarence C. Miner.
Paul Morris.
Burney V. Reany.
Robert S. Stewart.
Melvin C. Thomas.
Charles H. Wright.

land purchase control

Robert E. Frierson.
Paul F. Meredith.

drafting and mapping

Harry P. McKean.
Thomas Benson.
Paul C. Klyce.
Frank W. Ray.
Harry Tubis.

inspection and testing

Perry J. Freeman.
Frank W. Groh.
Edgar R. Kendall.
Franklin H. Stamps.
Charles D. Susano.

railroad engineering

Henry L. Fruend.
Robert H. Gleaves.
Glifford C. Muhs.
J. Butler Sullivan.

cemetery removal

Fred W. Allen.

utility relocation

H. Jervey Kelly.

DESIGN

Project designed by U. S. Bureau of Reclamation, with some participation by the following TVA forces:

Verne Gongwer, liaison officer

Civil and Structural Design

Ross M. Riegel.

dam and powerhouse

Erwin Maerker.
Cecil E. Pearce.

switchyard

John A. Howe.
Joseph C. Nowell, Jr.

caryville dam

Bernard R. Fuller.

big ridge dam

Donald H. Mattern.

Electrical Design

Raymond A. Hopkins.

dam and powerhouse

Edwin P. Almond.
Harry B. Barnhill.
Richard E. Behnke.
Joseph P. Kennell.

switchyard

Paul F. Galle.
Joseph P. Kennell.

Mechanical Design

William R. Chambers.

dam and powerhouse

Charles F. Ellis.
James M. Lloyd.
J. Frank Roberts.

switchyard

Carl B. Anderson.

Highway Engineering

James W. Bradner, Jr. (until October 1934).
Frank W. Webster (after October 1934).
James E. Moreland, asst. (after May 1936).

highway design

Francis E. Junior.

bridge design

Erwin Harsch.

field engineering

N. W. Dougherty (until September 1933).
Clarence L. Crabtree.
E. Dudley Jeffries.
Edgar F. McElfresh.
Sam F. Vesser.

CONSTRUCTION

Theodore B. Parker, chief construction engineer (after November 1935).
Ross White, supervising construction superintendent (after April 1935).

Engineering

Barton M. Jones, construction engineer.
C. Douglas Riddle, assistant (until October 1934).
Frederick A. Dale, assistant (October 1934 to July 1936).
Andrew M. Komora, assistant (after July 1936).

Office Engineering

Marshall P. Anderson (until November 1934).
Andrew M. Komora (July 1934 to July 1936).

Cost Engineering

Hendon R. Johnston (until May 1935).
Henry A. Sargent (after May 1935).
Rubie Kirby Parks, assistant (until January 1936).
Phillip A. Snedecor, assistant (August 1936 to April 1937).
Merton F. Ticknor, assistant (after March 1937).

Field Engineering

Hollis L. Broadfoot (until June 1936).
Ray M. Miller (after June 1936).

ENGINEERS AND PARTY CHIEFS

Charles A. Burnett.
Walter R. Carpenter.
John W. Clarke.
Edwin H. Hartman.
W. Douglas Lavers.
Ray M. Miller.
Crayton L. Norris.
Thomas F. Taylor.
Harry W. Watkins.

Concrete and Aggregates

Ivan L. Tyler.

AGGREGATE

Francisco Cadena.

SPECIAL INSTRUMENTS

Douglas H. McHenry.

Foundation Drilling and Grouting

James S. Lewis, Jr.
Gordon A. Carlson.
Jack C. Evans.

Electrical Engineering

Walter I. Self.

Construction

Ross White, construction superintendent (until April 1935).
Frederick C. Schlemmer, construction superintendent (after April 1935).

ASSISTANTS

Frederick C. Schlemmer (until April 1935).
Edwin M. Whipple (until January 1935).
Lex G. Phifer (January 1935 to July 19[illegible]
William A. Jones (after August 1936).

Mechanical

William A. Jones.
James L. Brown, assistant (until December 1934).
Harrison D. Irwin, assistant (after December 1934).

Electrical

Edward C. McClenagan.
Alex J. McKenzie, assistant (until February 1935).
Kay C. Christiansen, assistant (after August 1935).

Excavation and Quarry

Lex G. Phifer.

Carpentry

Peter M. Bedette.
William H. Mitchell, assistant.

Concreting

George E. Murphy.

Rigging

Ben T. Clark.

Accounting

Oscar A. Nystrom.

Storekeeping.

J. Burton Davis.

Timekeeping

Robert E. Jones.

CONSTRUCTION PLANT

Adolph J. Ackerman.

Robert T. Colburn. Philip H. Kline. Howard P. Maxton.

RESERVOIR CLEARANCE

Louis N. Allen.

Howard E. Davis. William R. Holden. Robert F. Holley.

MISCELLANEOUS CONSTRUCTION

Raymond H. Foss.
Leslie R. Ancill.
Lee M. Ragsdale.

CONSTRUCTION

Hayward McCarthy.

FIELD CLERICAL OFFICE

Anthony Passerine.

CARYVILLE DAM

John C. Burns.

HIGHWAYS AND BRIDGES

Ed M. Tate.

PROCUREMENT OF MATERIALS

Charles H. Garity.
Jack S. Beauchamp, assistant.
William J. Hagan, Jr., assistant.
Richard F. McLaughlin, assistant.
Administrative: Clifford L. Edington.

Contracts: John G. Werneke.
Inspection: Frank J. O'Brien.
Property: Addison E. Hook.
Purchasing: F. Leslie Lancaster.
Specifications: James H. Cheston.
Traffic: Louis B. Rockwell.
Warehouse and Stores: Rufus Holland.

LAND ACQUISITION

J. W. Cooper (until May 1934).
John I. Snyder (after May 1934).

BOARD OF REVIEW

Joe L. Burdette.
William P. Hemphill.
W. Edward Sanford.

TITLES

Leo L. Cole.
Michael F. Foley.
Roscoe Word.

APPRAISALS

William N. Garrett.

LAND BUYING

Edward W. Cowling, Jr.

LEGAL

James Lawrence Fly.
William C. Fitts, Jr.

Evans Dunn. Herbert S. Marks. Joseph C. Swidler.

FINANCE

Frank J. Carr.
Paul W. Ager, assistant.
Jerry F. Stone, adviser.

Treasurer:
Mrs. Florentine D. Goodrich (until January 1937).
J. Ed Campbell (after January 1937).

Auditing: Anson J. Robertson.
Construction Accounting: Fred L. Cavis.

General Accounting: Glenn P. Smith.
Plant Records: William J. Pollock.

LAND PLANNING AND HOUSING

Earle S. Draper.
Tracy B. Augur, assistant.

ARCHITECTURE

Roland A. Wank.

TOWN AND SITE PLANNING

Carroll A. Towne.

PARKS AND RECREATION

Sam F. Brewster.

RESERVOIR FAMILY REMOVAL

William G. Carnahan.
Marshall A. Wilson.

FORESTRY

Edward C. M. Richards.
Bernard Frank.

GENERAL OFFICE ENGINEER

Olaf Laurgaard (February 1936 to January 1937).
Harry Wiersema (after January 1937).

BUDGETS

Burgess B. Brier.
Albert B. Wilkinson.

REPORTS AND ORGANIZATION

George E. Tomlinson.

Earle B. Butler.
Thomas G. Harton.
Charles M. Turner.
Harold B. Vasey.

CONSULTANTS

ENGINEERING AND GEOLOGY

Charles P. Berkey.
O. N. Floyd.
L. C. Glenn.
George W. Hamilton.
L. F. Harza.
Arthur Keith.
C. H. Locher.
L. N. McClellan.
Warren J. Mead.
C. A. Paquette.
Charles H. Paul.
J. L. Savage.
F. W. Scheidenhelm.
C. M. Spofford.
A. C. Swinnerton.
William F. Uhl.
Sherman M. Woodward.

MINERAL DEPOSITS

Oliver Bowles.
E. F. Burchard.
J. J. Craig.
F. F. Graham.
E. A. Lewis.
Ernest West.
E. N. Williard.

RAILROAD ADJUSTMENTS

D. J. Brumley.
H. J. Saunders.
E. F. Wendt.

PERSONNEL

Floyd W. Reeves (until January 1936).
Gordon R. Clapp (after January 1936).
Arthur S. Jandrey, assistant (after April 1937).

CENTRAL OFFICE

Classification:
Milton V. Smith (until December 1936).
Carl L. Richey (after December 1936).
Employment:
Carl L. Richey until January 1936).
George Slover (after January 1936).
Labor relations: Clair C. Killen.
Personnel relations: Edwin B. Shultz.
Training:
J. Dudley Dawson (until February 1935).
Maurice Seay (after February 1935).

FIELD OFFICE

Gordon L. Jensen (until July 1936).
Page M. Darby (after July 1936).

OFFICE SERVICES

John F. Pierce.

HEALTH AND SAFETY

Eugene L. Bishop.
James A. Crabtree, assistant.

MALARIA CONTROL

Robert B. Watson.

SANITATION

Walter G. Stromquist.

SAFETY

Paul F. Stricker (until May 1935).
Donald F. McMurchy (after May 1935).
Construction safety: Henry J. Fulmer.
Public safety: Kenneth A. Rouse.

MEDICAL STAFF

William D. Strayhorn, Jr.
Dam:
Manly F. Langston.
S. Fred Strain.
Robert B. Watson.
Reservoir:
Eugene B. Glenn.
Glenn D. Grubb.
H. Stirl Rule.
H. L. C. Wilkerson.

ARCHAEOLOGY

William S. Webb. T. Levron Howard.

OPERATIONS

A. H. Sullivan.

C. L. Karr. W. W. Woodruff. E. E. Robinson.

Labor classifications and hourly rates of pay, Norris project

Unless otherwise indicated, rates shown were for all types of construction. Rates for "dam" applied to only the dam structures, while "Miscellaneous" rates applied to such other work as buildings, highways, and railroads.

Classification	Prior to Dec. 1, 1933	Effective Dec. 1, 1933	In effect Oct. 29, 1935 [1]	Effective Feb. 1, 1936	Effective Jan. 1, 1937
Apprentice and helpers, dam	$0.30	$0.60–$0.75	$0.60–$0.75	$0.60–$0.75	$0.60–$0.75
Miscellaneous	.30	.55	.60– .75	.60– .75	.60– .75
Asbestos worker (heat insulation)				1.00	1.10
Blacksmith	.80	1.00	1.00	1.00	1.10
Boilermaker		1.00	1.00	1.00	1.10
Building trades, unclassified		1.00	1.00	1.00	1.10
Cableway operator				1.50	1.50
Carpenter (form and finish)	.75	1.00	1.00	1.00	1.10
Compressor operator, dam	.70	.60–.75	.60– .75	.60– .75	.60– .75
Miscellaneous	.70	.80	.60– .75	.60– .75	.60– .75
Heavy duty compressor operator					1.00
Concrete finisher	.60	1.00	1.00	1.00	1.10
Concrete mixer operator, dam	.70	.75	.75	.60– .75	.60– .75
Miscellaneous	.70	.80	.75	.60– .75	.60– .75
Concrete puddler		.45	.55	.55	.55
Concrete rodder and spreader		.75			
Core drill operator		1.00	1.00	1.00	1.00
Crane or derrick operator, dam	1.00	1.50	1.50	1.50	1.25– 1.50
Miscellaneous	1.00	1.25	1.25	1.25	1.25– 1.50
Dinkey operator		.75	.75	.75	.75
Drill-dresser machine operator		1.00	1.00	1.00	1.10
Electrician	.80	1.00	1.00	1.00	1.10
Elevating grader operator				1.00	1.00
Fireman		.75	.75	.75	.75
Flagman				.60	.60
Foremen—Labor	1.00	[2] .80 [2] 1.00	[2] .90 [2] 1.00	1.00	1.10
Skilled trades		1.25	[2] 1.10 [2] 1.25	[3] 1.25 [3] 1.375	[4] 1.25 [4] 1.35 [4] 1.50
Foremen, sub—Labor	.75		.75	.75	.75
Skilled trades				[3] 1.10 [3] 1.20	[4] 1.10 [4] 1.20 [4] 1.35
Glaziers				1.00	1.10
Hoist operator—2 drums		1.00	1.00	1.00	1.10
1 drum		1.00	1.00	1.00	1.00
Iron worker, ornamental				1.125	1.25
Jackhammer operator, dam	.30	.60	.60	.60	.60
Miscellaneous	.30	.55	.60	.60	.60

[1] Between December 1933 and October 1935, rates for individual classifications were changed as such changes were considered appropriate. All changes after October 1935 became effective on the dates shown.
[2] Rates shown for 8-hour and 5½- to 6-hour shifts respectively.
[3] Rates shown for supervising trades with $1 and $1.125 rates respectively.
[4] Rates shown for supervising trades with $1, $1.10, and $1.25 rates, respectively.

Labor classifications and hourly rates of pay, Norris project—Continued

Classification	Prior to Dec. 1, 1933	Effective Dec. 1, 1933	In effect Oct. 29, 1935	Effective Feb. 1, 1936	Effective Jan. 1, 1937
Labor, unclassified	$0.30	$0.45	$0.45	$0.45	$0.45
Lathers	-----	1.00	1.10	1.00	1.10
Le-Tourneau operator	-----	-----	-----	-----	1.00
Lineman and armature winder	-----	-----	-----	1.00	1.10
Locomotive operator	-----	1.00	1.00	1.00	1.10
Machinist and general mechanic	-----	1.00	1.00	1.00	1.10
Marine engineer—50 tons or over	-----	1.00	1.00	1.00	1.10
Less than 50 tons	-----	1.00	1.00	1.00	1.00
Marine pilot—50 tons or over	-----	-----	1.00	1.00	1.10
Less than 50 tons	-----	-----	1.00	1.00	1.00
Masons, brick and stone	.90	1.00	1.00	1.125	1.25
Mason tender	.35	.55	.60	.60	.60
Millwright	-----	1.00	1.00	1.00	1.10
Mortar mixer	.35	.55	.60	.60	.75
Motorboat and tugboat operator	-----	.60– .75	-----	.75	.60– .75
Nozzlemen—Sluicing	-----	.60	.60	-----	-----
Oiler	-----	.60	.60	.60	.60– .75
Painters and decorators	.70	1.00	1.00	1.00	1.10
Painters, sign	-----	-----	-----	1.00	1.10
Pile driver	-----	1.00	1.00	1.00	1.10
Pipefitter	-----	1.00	1.10	1.00	1.25
High pressure permanent	-----	-----	-----	1.125	1.25
Plasterers	-----	1.00	1.10	1.00	1.25
Plumber and steamfitter	1.00	1.00	1.10	1.125	1.25
Powder man	-----	.75	.75	.75	.75
Pump operator, dam	.70	.60	.60	.60– .75	.60– .75
Miscellaneous	.70	.80	.60	.60– .75	.60– .75
Rigger	-----	1.00	1.00	1.00	1.25
Road machine operator	.55	.80	.75	.75	.75
Road roller operator	.60	.80	.75	.75	.75
Roofer	.75	1.00	1.00	1.00	1.10
Saw filer	-----	1.00	1.00	1.00	1.10
Sewer layer	.30	.55	.60	.60	.60
Shaft and tunnel miner	-----	.75	.75	.75	.75
Sheet metal worker	-----	1.00	1.00	1.00	1.10
Shingle splitter	.60	.75	.75	-----	-----
Shovel or dragline operator:					
Over ¾ yard, dam	1.00	1.50	1.50	1.50	1.50
¾ yard and under, dam	1.00	1.50	1.50	1.50	1.25
Miscellaneous	1.00	1.30	1.25	1.25	1.25– 1.50
Signalman (cableway)	-----	-----	-----	1.00	1.25
Steam engine operator	-----	1.00	1.00	1.00	-----
Steel worker, reinforcing (Bending, placing, tying)	-----	.75	1.00	1.00	1.00
Steel worker, structural (erecting, riveting, heating)	-----	1.00	1.10	1.125	1.25
Teamster	-----	-----	.45– .60	.45– .60	.45– .60
Tool dresser	-----	1.00	1.00	1.00	1.10
Tractor and grader operator, dam	.60	.75	.75	.75	.75
Miscellaneous	.60	.80	.75	.75	.75
Trenching machine operator	1.00	1.25	1.50	1.00	1.10
Truck operator:					
All sizes, miscellaneous	.30	.55	-----	-----	-----
4-ton, dam	.30	.75	-----	-----	-----
1½-ton, dam	.30	.60	-----	-----	-----
Over 5-ton	-----	-----	.75	.75	.75
5-ton and under	-----	-----	.60	.60	.60
Wagon drill operator	-----	.60– .75	.75	.75	.75
Watchman	-----	.45	.45	.45	.45
Welder	-----	1.00	1.00	1.00	1.25
For certified welding	-----	-----	-----	1.125	-----

RESERVOIR CLEARANCE

Classification	Prior to Dec. 1, 1933	Effective Dec. 1, 1933	In effect Oct. 29, 1935	Effective Feb. 1, 1936	Effective Jan. 1, 1937
Foremen—Labor	-----	-----	$0.85	$0.85	$1.00
Foremen, sub—Labor	-----	$0.625	.75	.75	.75
Labor:					
Skilled	-----	.85	-----	-----	-----
Semiskilled	-----	.50–$0.625	.625	.625	-----
Unskilled	-----	.375	.45	.45	.45
Log scaler	-----	.625	-----	-----	-----
Machine operator	-----	-----	.85	.85	.85
Motorboat and tugboat operator	-----	-----	.625	.625	-----
Saw filer	-----	.625	.70	.70	.75
Tallyman	-----	-----	.625	.625	-----
Timber rigger	-----	-----	.625	.625	.625
Tree climber	-----	-----	.625	.625	-----
Truck driver	-----	-----	.60	.60	-----

EMPLOYEE RELATIONSHIP POLICY

The employee relationship policy as adopted by the Board and published herewith is the result of long study and discussion in which both employees and management participated. In a real sense, the policy is a product of cooperative and collective effort and suggestion. The series of conferences conducted prior to formulation of the final draft enlisted the suggestions and criticisms from a large number of employees, individually and through their duly authorized representatives. This is essentially a policy of the Board of Directors. The Board, however, takes pleasure in the fact that the policy was drawn in its final form after collective conference and understanding.

INTRODUCTORY STATEMENT

The Board of Directors of the Tennessee Valley Authority deems it advisable to formulate a policy governing the relationship between employees and management and to provide machinery for its administration. In the formulation of this policy the Authority recognizes that it is an agency of the sovereign Government of the United States. As a consequence, the employee relationship policy of the Authority must conform to national policy and the Federal Government must be in final control. In this respect the Authority differs from a private employer. Subject to these conditions, the Authority can establish governing principles upon which a progressive program of employee relations may be based. This statement represents an initial effort to formulate a labor policy for the Tennessee Valley Authority. It will undoubtedly require modification from time to time, and it should be regarded, not as fixed and unchangeable, but as subject to growth and change in the light of experience.

The Authority will support as favorable labor standards and employment conditions as are consistent with the national welfare, having regard for the fact that the work of the Authority is being financed initially by the people of the United States.

The employee relationship policy of the Authority will find detailed expression in specific rules and regulations governing labor standards, rates of pay, classification, hours of work, and employment conditions as well as in memoranda of understanding. Such rules, regulations, and understandings as are now in effect or as may hereafter be adopted, will be published for the information and guidance of all concerned.

THE POLICY

1. An obligation rests upon every member of the management and supervisory staff, as well as upon each supervised employee of the Tennessee Valley Authority, to render honest, efficient, and economical service in the performance of his duties. Organizations and associations of supervised employees and of the supervisory and management staff are likewise subject to this obligation.

2. Members of the management and supervisory staff and the supervised employees of the Tennessee Valley Authority together comprise an organization for public service. The whole-hearted cooperation of all members of the organization in carrying out this policy and the Tennessee Valley Authority Act is essential to attain the objectives of the Authority.

3. For the purposes of collective bargaining and employee-management cooperation, employees of the Authority shall have the right to organize and designate representatives of their own choosing. In the exercise of this right they shall be free from any and all restraint, interference, or coercion on the part of the management and supervisory staff. This paragraph shall not be construed to limit the rights of employees to organize for other lawful purposes.

4. No employee of the Authority and no one seeking employment shall be required as a condition of employment, transfer, promotion, or retention in service to join or to refrain from joining any organization or association of employees.

5. There shall be no discrimination against representatives of employees of the Authority nor shall employees suffer discrimination because of membership or nonmembership in any organization or association of employees.

6. The majority of the employees as a whole, or of any professional group. or craft, or other appropriate unit, shall have the right to determine the organization, person or persons who shall represent the employees as a whole, or

any such professional group, or craft, or unit. If a dispute shall arise among employees as to who are the duly authorized representatives of the employees involved, the Personnel Division of the Authority, which shall include a labor relations staff, shall investigate such dispute and attempt to adjust it on its merits. In the adjustment of such a dispute the Personnel Division may, if the parties agree in writing, conduct an election to determine the duly authorized representatives of the employees involved. In the conduct of such an election the Personnel Division shall designate the persons who may participate. Should the Personnel Division be unable to adjust the dispute either party shall be free to invoke the services of the National Labor Relations Board.

7. Disputes between an employee and the management growing out of grievances or out of the interpretation or application of the published rules and regulations of the Authority governing labor standards, rates of pay, classification, hours of work, employment conditions, and the like, shall be handled by the employee or his representative through established supervisory channels, up to and including the designated chief supervisory officer concerned, as defined from time to time by the proper administrative officers. Failing prompt and satisfactory adjustment, the employee or his representative may appeal the dispute to the central office of the Personnel Division for investigation and adjustment.

8. Hourly and annual rates of pay shall be determined on the basis of occupational classification to assure comparable rates for comparable work. Schedules of such rates shall be published and made available to all employees. The division of occupations into classes of work shall give due and adequate recognition to intelligence, skill, training, and experience. The classification of occupations into classes and grades of work need not be bound by traditional rules and customs. The allocation of jobs or positions to scheduled grades shall be upon the basis of the duties to be performed. In the classification of annually rated positions due regard will be given to standards of classification and rates of pay prevailing in the classified Federal service. No discrimination in occupational classification or in rates of pay shall be made on the basis of sex or race.

9. The regular hours of employment of all hourly rated employees shall be bulletined by kinds of employment or by services at each place of employment. Such bulletins shall indicate how these hours may be worked in any 24-hour period. One, two, three, or four shifts may be worked on any kind of employment or service at each place of employment. The regular hours of employment shall not exceed 8 in any 24-hour period. All work of hourly rated employees shall be so organized as to provide at least 1 day's rest in 7. Whenever feasible such day of rest shall be Sunday. All authorized overtime worked in excess of 8 hours in any 1 working day shall be paid for at the rate of time and one-half. Any hourly rated employee required to work on Memorial Day, Fourth of July, Labor Day, Thanksgiving Day, Christmas, or on his designated day of rest, shall be paid at the rate of time and one-half. The supervisory and management staff and the supervised employees are expected, as a matter of good management and efficiency, to make every possible effort to minimize overtime and to conform to the schedule of bulletined hours. Failure to obtain reasonable compliance with the schedule of bulletined hours to assure elimination of excessive overtime will be interpreted by the Board of Directors as indicative of inefficient supervision and workmanship.

10. The regular hours of employment of all annually rated employees shall be bulletined by kinds of employment or by services at each place of employment. The regular hours of employment shall not exceed 8 in any 24-hour period. All work of annually rated employees shall be so organized as to provide at least 1 day's rest in 7. Whenever possible such day of rest shall be Sunday. All authorized overtime worked either before or after regular bulletined hours shall be recorded and accumulated as earned annual leave, to be taken in accordance with the regulations governing annual leave. Any annually rated employee required to work on Memorial Day, Fourth of July, Labor Day, Thanksgiving Day, Christmas, or on his designated day of rest, shall be allowed to accumulate such time as earned annual leave to be taken in accordance with the regulations governing annual leave.

11. During periods of marked unemployment, hours of work will be kept as few as consistent with efficiency in production and reasonable minimum income.

12. The desirability of giving advance notice before reducing forces is recognized.

13. Appointments to the service of the Authority will be made on the basis of merit and efficiency as determined by such factors as intelligence, ability, skill, training, and experience. Promotion, demotion, transfer, retention in, or termination of, service with the Authority will be made on the basis of merit and efficiency, having due regard for length of service. No test of political belief or affiliation will be required of any employee or considered in his appointment, promotion, demotion, transfer, retention in, or termination of, service with the Authority. If an employee, who is doing the best work he can in good spirit, is found to be unsuited for the tasks to which he is assigned, an earnest effort shall be made to place him at other work for which he is better suited. Because it is in accordance with sound public policy that all work shall be done by those who can and will do it best, it shall be the policy of the Tennessee Valley Authority to add to its staff those who will distinctly raise its standards of work, and to offer equal opportunities for raising standards to employees already in the service. Employment in a position is not a vested right to be retained primarily because of possession, but only if quality of service justifies continuance of employment.

14. No person under 16 years of age shall be employed by the Authority. No person under 18 years of age shall be employed by the Authority in a hazardous occupation.

15. Appointments to the service will not be made when such appointments involve nepotism as defined by the Board of Directors of the Authority.

16. Supervisors may, for just cause, terminate the service of any employee under their supervision, and such termination shall separate the employee from pay status. In so doing, the supervisor shall state the cause for termination in writing. A copy of the written notice stating such cause shall be sent to the Personnel Division and to the employee upon request. No employee shall be discharged from the Authority, however, without the approval of the Personnel Division subsequent to a fair hearing if requested by the employee or his representative within 10 days of the effective date of termination.

17. Adequate personnel and service records shall be kept for every employee in order that recorded data may serve as a basis for appraisal of merit and efficiency.

18. The Authority will endeavor to make adequate provision for the safety and health of employees at their places of employment. The management and supervisory staff will endeavor to place employees in such tasks as fall within the limits of their physical powers, so far as these can be reasonably ascertained.

19. In accordance with section 3 of the act creating the Authority not less than the rate of wages for work of a similar nature prevailing in the vicinity shall be paid to laborers and mechanics. In the event any question arises as to what are the prevailing rates of wages, which question cannot be settled by conference between the duly authorized representatives of the employees and the Authority, it shall be referred to the Secretary of Labor for determination, and the decision of the Secretary shall be final. In the determination of such prevailing rate or rates, due regard shall be given to those rates which have been secured through collective agreement by representatives of employers and employees.

20. All contracts to which the Authority is a party and which required the employment of laborers and mechanics in the construction, alteration, maintenance, or repair of buildings, dams, locks, or other projects shall contain a provision that not less than the prevailing rate of wages for work of a similar nature prevailing in the vicinity, shall be paid to such laborers or mechanics.

21. Schedules of rates of pay, whether hourly or annual, shall be published and remain in force and effect until revised or modified as provided herein. Schedules in effect will be open for revision not more often than once each calendar year. Proposed revisions will be investigated and studied by the Personnel Division in conference with supervisors and employees or their duly authorized representatives. Any requests for revisions of established rates of pay submitted during the year and not later than November 1, will be considered and acted upon by the Board of Directors of the Authority before the end of the calendar year. Published schedules of rates of pay shall designate the minimum rate for hourly employees and the minimum rate for annual employees below which no occupation will be classified. Provision may be made for special rates of pay for partially disabled persons or for intermittent service.

22. Rules and regulations defining labor standards and conditions of employment, generally applicable to the employees of the Authority, other than rates

of pay and occupational classifications, will be adopted, modified, or amended from time to time and thereupon published. At least 30 days' published notice shall be given of any proposed new rule or change in established rules. No new rule may be adopted or existing rule changed until the duly authorized representatives of employees have had reasonable opportunity to confer with the supervisory staff and the Personnel Division.

CONCLUDING STATEMENT

As a further development of this policy the Board of Directors looks forward to the establishment of joint conferences between the duly authorized representatives of the supervised employees and the supervisory and management staff for the purpose of systematic employee-management cooperation. The Board recognizes that responsible organizations and associations of employees are helpful to such cooperation. It is suggested that such joint cooperative conferences might well devote themselves to furthering the objectives for which the Tennessee Valley Authority was created. In so doing these conferences might consider such matters as the elimination of waste in construction and production; the conservation of materials, supplies, and energy; the improvement in quality of workmanship and services; the promotion of education and training; the correction of conditions [illegible] grievances and misunderstandings; the encouragement of courtesy in the relations of employees with the public; the safeguarding of health; the prevention of hazards to life and property; the betterment of employment conditions; and the strengthening of the morale of the service. In the achievement of these objectives, it will not be desirable for these cooperative conferences to attempt to adjust individual disputes either among employees or between employees and their supervisors, it being the intent of this policy to adjust these matters promptly as elsewhere provided. When the other features of this policy are satisfactorily translated into practice, the Board will stand ready to consider plans prepared jointly by supervised employees and the management by means of which these joint conferences may be established.

Signed:

Arthur E. Morgan, *Chairman.*
Harcourt A. Morgan.
David E. Lilienthal.

Approved August 28, 1935.

Appendix J

[Public—No. 17—73d Congress]

[H. R. 5081]

[As amended by Public—No. 412—74th Congress]

[H. R. 8632]

AN ACT To improve the navigability and to provide for the flood control of the Tennessee River; to provide for reforestation and the proper use of marginal lands in the Tennessee Valley; to provide for the agricultural and industrial development of said valley; to provide for the national defense by the creation of a corporation for the operation of Government properties at and near Muscle Shoals in the State of Alabama, and for other purposes

Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That for the purpose of maintaining and operating the properties now owned by the United States in the vicinity of Muscle Shoals, Alabama, in the interest of the national defense and for agricultural and industrial development, and to improve navigation in the Tennessee River and to control the destructive flood waters in the Tennessee River and Mississippi River Basins, there is hereby created a body corporate by the name of the "Tennessee Valley Authority" (hereinafter referred to as the "Corporation"). The board of directors first appointed shall be deemed the incorporators, and the incorporation shall be held to have been effected from the date of the first meeting of the board. This Act may be cited as the "Tennessee Valley Authority Act of 1933."

Sec. 2. (a) The board of directors of the Corporation (hereinafter referred to as the "board") shall be composed of three members, to be appointed by the President, by and with the advice and consent of the Senate. In appointing the members of the board, the President shall designate the chairman. All other officials, agents, and employees shall be designated and selected by the board.

(b) The terms of office of the members first taking office after the approval of this Act shall expire as designated by the President at the time of nomination, one at the end of the third year, one at the end of the sixth year, and one at the end of the ninth year, after the date of approval of this Act. A successor to a member of the board shall be appointed in the same manner as the original members and shall have a term of office expiring nine years from the date of the expiration of the term for which his predecessor was appointed.

(c) Any member appointed to fill a vacancy in the board occurring prior to the expiration of the term for which his predecessor was appointed shall be appointed for the remainder of such term.

(d) Vacancies in the board so long as there shall be two members in office shall not impair the powers of the board to execute the functions of the Corporation, and two of the members in office shall constitute a quorum for the transaction of the business of the board.

(e) Each of the members of the board shall be a citizen of the United States and shall receive a salary at the rate of $10,000 a year, to be paid by the Corporation as current expenses. Each member of the board, in addition to his salary, shall be permitted to occupy as his residence one of the dwelling houses owned by the Government in the vicinity of Muscle Shoals, Alabama, the same to be designated by the President of the United States. Members of the board shall be reimbursed by the Corporation for actual expenses (including traveling and subsistence expenses) incurred by them in the performance of the duties vested in the board by this Act. No member of said board shall, during his continuance in office, be engaged in any other business, but each member shall devote himself to the work of the Corporation.

(f) No director shall have financial interest in any public-utility corporation engaged in the business of distributing and selling power to the public nor in any corporation engaged in the manufacture, selling, or distribution of fixed nitrogen or fertilizer, or any ingredients thereof, nor shall any member have any interest in any business that may be adversely affected by the success of the Corporation as a producer of concentrated fertilizers or as a producer of electric power.

(g) The board shall direct the exercise of all the powers of the Corporation.

(h) All members of the board shall be persons who profess a belief in the feasibility and wisdom of this Act.

SEC. 3. The board shall, without regard to the provisions of civil-service laws applicable to officers and employees of the United States, appoint such managers, assistant managers, officers, employees, attorneys, and agents as are necessary for the transaction of its business, fix their compensation, define their duties, require bonds of such of them as the board may designate, and provide a system of organization to fix responsibility and promote efficiency. Any appointee of the board may be removed in the discretion of the board. No regular officer or employee of the Corporation shall receive a salary in excess of that received by the members of the board.

All contracts to which the Corporation is a party and which require the employment of laborers and mechanics in the construction, alteration, maintenance, or repair of buildings, dams, locks, or other projects shall contain a provision that not less than the prevailing rate of wages for work of a similar nature prevailing in the vicinity shall be paid to such laborers or mechanics.

In the event any dispute arises as to what are the prevailing rates of wages, the question shall be referred to the Secretary of Labor for determination, and his decision shall be final. In the determination of such prevailing rate or rates due regard shall be given to those rates which have been secured through collective agreement by representatives of employers and employees.

Where such work as is described in the two preceding paragraphs is done directly by the Corporation the prevailing rate of wages shall be paid in the same manner as though such work had been let by contract.

Insofar as applicable, the benefits of the Act entitled "An Act to provide compensation for employees of the United States suffering injuries while in the performance of their duties, and for other purposes," approved September 7, 1916, as amended, shall extend to persons given employment under the provisions of this Act.

SEC. 4. Except as otherwise specifically provided in this Act, the Corporation—

(a) Shall have succession in its corporate name.

(b) May sue and be sued in its corporate name.

(c) May adopt and use a corporate seal, which shall be judicially noticed.

(d) May make contracts, as herein authorized.

(e) May adopt, amend, and repeal bylaws.

(f) May purchase or lease and hold such real and personal property as it deems necessary or convenient in the transaction of its business, and may dispose of any such personal property held by it.

The board shall select a treasurer and as many assistant treasurers as it deems proper, which treasurer and assistant treasurers shall give such bonds for the safekeeping of the securities and moneys of the said Corporation as the board may require: *Provided*, That any member of said board may be removed from office at any time by a concurrent resolution of the Senate and the House of Representatives.

(g) Shall have such powers as may be necessary or appropriate for the exercise of the powers herein specifically conferred upon the Corporation.

(h) Shall have power in the name of the United States of America to exercise the right of eminent domain, and in the purchase of any real estate or the acquisition of real estate by condemnation proceedings, the title to such real estate shall be taken in the name of the United States of America, and thereupon all such real estate shall be entrusted to the Corporation as the agent of the United States to accomplish the purposes of this Act.

(i) Shall have power to acquire real estate for the construction of dams, reservoirs, transmission lines, powerhouses, and other structures, and navigation projects at any point along the Tennessee River, or any of its tributaries, and in the event that the owner or owners of such property shall fail and refuse to sell to the Corporation at a price deemed fair and reasonable by

the board, then the Corporation may proceed to exercise the right of eminent domain, and to condemn all property that it deems necessary for carrying out the purposes of this Act, and all such condemnation proceedings shall be had pursuant to the provisions and requirements hereinafter specified, with reference to any and all condemnation proceedings: *Provided*, That nothing contained herein or elsewhere in this Act shall be construed to deprive the Corporation of the rights conferred by the Act of February 26, 1931 (46 Stat. 1422, ch. 307, secs. 1 to 5, inclusive), as now compiled in sections 258a to 258e, inclusive, of title 40 of the United States Code.

[(j) Shall have power to construct dams, reservoirs, powerhouses, power structures, transmission lines, navigation projects, and incidental works in the Tennessee River and its tributaries, and to unite the various power installations into one or more systems by transmission lines.]

(j) Shall have power to construct such dams and reservoirs in the Tennessee River and its tributaries, as in conjunction with Wilson Dam, and Norris, Wheeler, and Pickwick Landing Dams, now under construction, will provide a nine-foot channel in the said river and maintain a water supply for the same, from Knoxville to its mouth, and will best serve to promote navigation on the Tennessee River and its tributaries and control destructive flood waters in the Tennessee and Mississippi River drainage basins; and shall have power to acquire or construct powerhouses, power structures, transmission lines, navigation projects, and incidental works in the Tennessee River and its tributaries and to unite the various power installations into one or more systems by transmission lines. The directors of the Authority are hereby directed to report to Congress their recommendations not later than April 1, 1936, for the unified development of the Tennessee River system.

(k) At any time before the expiration of five years from the date when this section, as amended, becomes law may in the name of and as agent for the United States and subject to approval of the President, dispose of any of such real property as in the judgment of the board may be no longer necessary in carrying out the purposes of this Act, but no land shall be conveyed on which there is a permanent dam, hydraulic power plant, fertilizer plant, or munitions plant, heretofore or hereafter built by or for the United States or for the Authority.

(l) Shall have power to advise and cooperate in the readjustment of the population displaced by the construction of dams, the acquisition of reservoir areas, the protection of watersheds, the acquisition of rights-of-way, and other necessary acquisitions of land, in order to effectuate the purposes of the Act; and may cooperate with Federal, state, and local agencies to that end.

SEC. 5. The board is hereby authorized—

(a) To contract with commercial producers for the production of such fertilizers or fertilizer materials as may be needed in the Government's program of development and introduction in excess of that produced by Government plants. Such contracts may provide either for outright purchase of materials by the board or only for the payment of carrying charges on special materials manufactured at the board's request for its program.

(b) To arrange with farmers and farm organizatons for large-scale practical use of the new forms of fertilizers under conditions permitting an accurate measure of the economic return they produce.

(c) To cooperate with National, State, district, or county experimental stations or demonstration farms, with farmers, landowners, and associations of farmers or landowners, for the use of new forms of fertilizer or fertilizer practices during the initial or experimental period of their introduction, and for promoting the prevention of soil erosion by the use of fertilizers and otherwise.

Material added by the 1935 amendment is underlined.
Material deleted by the 1935 amendment is enclosed in brackets.

(d) The board, in order to improve and cheapen the production of fertilizer, is authorized to manufacture and sell fixed nitrogen, fertilizer, and fertilizer ingredients at Muscle Shoals by the employment of existing facilities, by modernizing existing plants, or by any other process or processes that in its judgment shall appear wise and profitable for the fixation of atmospheric nitrogen or the cheapening of the production of fertilizer.

(e) Under the authority of this Act the board may make donations or sales of the product of the plant or plants operated by it to be fairly and equitably distributed through the agency of county demonstration agents, agricultural colleges, or otherwise as the board may direct, for experimentation, education, and introduction of the use of such products in cooperation with practical farmers so as to obtain information as to the value, effect, and best methods of their use.

(f) The board is authorized to make alterations, modifications, or improvements in existing plants and facilities, and to construct new plants.

(g) In the event it is not used for the fixation of nitrogen for agricultural purposes or leased, then the board shall maintain in stand-by condition nitrate plant numbered 2, or its equivalent, for the fixation of atmospheric nitrogen, for the production of explosives in the event of war or a national emergency, until the Congress shall by joint resolution release the board from this obligation, and if any part thereof be used by the board for the manufacture of phosphoric acid or potash, the balance of nitrate plant numbered 2 shall be kept in stand-by condition.

(h) To establish, maintain, and operate laboratories and experimental plants, and to undertake experiments for the purpose of enabling the Corporation to furnish nitrogen products for military purposes, and nitrogen and other fertilizer products for agricultural purposes in the most economical manner and at the highest standard of efficiency.

(i) To request the assistance and advice of any officer, agent, or employee of any executive department or of any independent office of the United States, to enable the Corporation the better to carry out its powers successfully, and as far as practicable shall utilize the services of such officers, agents, and employees, and the President shall, if in his opinion the public interest, service, or economy so require, direct that such assistance, advice, and service be rendered to the Corporation, and any individual that may be by the President directed to render such assistance, advice, and service shall be thereafter subject to the orders, rules, and regulations of the board: *Provided*, That any invention or discovery made by virtue of and incidental to such service by an employee of the Government of the United States serving under this section, or by any employee of the Corporation, together with any patents which may be granted thereon, shall be the sole and exclusive property of the Corporation, which is hereby authorized to grant such licenses thereunder as shall be authorized by the board: *Provided further*, That the board may pay to such inventor such sum from the income from sale of licenses as it may deem proper.

(j) Upon the requisition of the Secretary of War or the Secretary of the Navy to manufacture for and sell at cost to the United States explosives or their nitrogenous content.

(k) Upon the requisition of the Secretary of War, the Corporation shall allot and deliver without charge to the War Department so much power as shall be necessary in the judgment of said Department for use in operation of all locks, lifts, or other facilities in aid of navigation.

(l) To produce, distribute, and sell electric power as herein particularly specified.

(m) No products of the Corporation shall be sold for use outside of the United States, its Territories, and possessions, except to the United States Government for the use of its Army and Navy, or to its allies in case of war.

(n) The President is authorized, within twelve months after the passage of this Act, to lease to any responsible farm organization, or to any corporation organized by it, nitrate plant numbered 2 and Waco Quarry, together with the railroad connecting said quarry with nitrate plant numbered 2, for a term not exceeding fifty years at a rental of not less than $1 per year, but such authority shall be subject to the express condition that the lessee shall use said property during the term of said lease exclusively for the manufacture of fertilizer and fertilizer ingredients to be used only in the manufac-

ture of fertilizer by said lessee and sold for use as fertilizer. The said lessee shall covenant to keep said property in first-class condition, but the lessee shall be authorized to modernize said plant numbered 2 by the installation of such machinery as may be necessary, and is authorized to amortize the cost of said machinery and improvements over the term of said lease or any part thereof. Said lease shall also provide that the board shall sell to the lessee power for the operation of said plant at the same schedule of prices that it charges all other customers for power of the same class and quantity. Said lease shall also provide that, if the said lessee does not desire to buy power of the publicly owned plant, it shall have the right to purchase its power for the operation of said plant of the Alabama Power Company or any other publicly or privately owned corporation engaged in the generation and sale of electric power, and in such case the lease shall provide further that the said lessee shall have a free right-of-way to build a transmission line over Government property to said plant, paying the actual expenses and damages, if any, incurred by the Corporation on account of such line. Said lease shall also provide that the said lessee shall covenant that during the term of said lease the said lessee shall not enter into any illegal monopoly, combination, or trust with any privately owned corporation engaged in the manufacture, production, and sale of fertilizer with the object or effect of increasing the price of fertilizer to the farmer.

SEC. 6. In the appointment of officials and the selection of employees for said Corporation, and in the promotion of any such employees or officials, no political test or qualification shall be permitted or given consideration, but all such appointments and promotions shall be given and made on the basis of merit and efficiency. Any member of said board who is found by the President of the United States to be guilty of a violation of this section shall be removed from office by the President of the United States, and any appointee of said board who is found by the board to be guilty of a violation of this section shall be removed from office by said board.

SEC. 7. In order to enable the Corporation to exercise the powers and duties vested in it by this Act—

(a) The exclusive use, possession, and control of the United States nitrate plants numbered 1 and 2, including steam plants, located, respectively, at Sheffield, Alabama, and Muscle Shoals, Alabama, together with all real estate and buildings connected therewith, all tools and machinery, equipment, accessories, and materials belonging thereto, and all laboratories and plants used as auxiliaries thereto; the fixed-nitrogen research laboratory, the Waco limestone quarry, in Alabama, and Dam Numbered 2, located at Muscle Shoals, its powerhouse, and all hydroelectric and operating appurtenances (except the locks), and all machinery, lands, and buildings in connection therewith, and all appurtenances thereof, and all other property to be acquired by the Corporation in its own name or in the name of the United States of America, are hereby intrusted to the Corporation for the purposes of this Act.

(b) The President of the United States is authorized to provide for the transfer to the Corporation of the use, possession, and control of such other real or personal property of the United States as he may from time to time deem necessary and proper for the purposes of the Corporation as herein stated.

SEC. 8. (a) The Corporation shall maintain its principal office in the immediate vicinity of Muscle Shoals, Alabama. The Corporation shall be held to be an inhabitant and resident of the northern judicial district of Alabama within the meaning of the laws of the United States relating to the venue of civil suits.

(b) The Corporation shall at all times maintain complete and accurate books of accounts.

(c) Each member of the board, before entering upon the duties of his office, shall subscribe to an oath (or affirmation) to support the Constitution of the United States and to faithfully and impartially perform the duties imposed upon him by this Act.

SEC. 9. (a) The board shall file with the President and with the Congress, in December of each year, a financial statement and a complete report as to the business of the Corporation covering the preceding governmental fiscal year. This report shall include an itemized statement of the cost of power at each power station, the total number of employees and the names, salaries, and duties of those receiving compensation at the rate of more than $1,500 a year.

(b) All purchases and contracts for supplies or services, except for personal services, made by the Corporation, shall be made after advertising, in such manner and at such times sufficiently in advance of opening bids, as the board shall determine to be adequate to insure notice and opportunity for competition: *Provided*, That advertisement shall not be required when, (1) an emergency requires immediate delivery of the supplies or performance of the services; or (2) repair parts, accessories, supplemental equipment, or services are required for supplies or services previously furnished or contracted for; or (3) the aggregate amount involved in any purchase of supplies or procurement of services does not exceed $500; in which cases such purchases of supplies or procurement of services may be made in the open market in the manner common among business men: *Provided further*, That in comparing bids and in making awards the board may consider such factors as relative quality and adaptability of supplies or services, the bidder's financial responsibility, skill, experience, record of integrity in dealing, ability to furnish repairs and maintenance services, the time of delivery or performance offered, and whether the bidder has complied with the specifications.

The Comptroller General of the United States shall audit the transactions of the Corporation at such times as he shall determine, but not less frequently than once each governmental fiscal year, with personnel of his selection. In such connection, he and his representatives shall have free and open access to all papers, books, records, files, accounts, plants, warehouses, offices, and all other things, property, and places belonging to or under the control of or used or employed by the Corporation, and shall be afforded full facilities for counting all cash and verifying transactions with and balances in depositaries. He shall make report of each such audit in quadruplicate, one copy for the President of the United States, one for the chairman of the board, one for public inspection at the principal office of the Corporation, and the other to be retained by him for the uses of the Congress: *Provided*, That such report shall not be made until the Corporation shall have had reasonable opportunity to examine the exceptions and criticisms of the Comptroller General of the General Accounting Office, to point out errors therein, explain or answer the same, and to file a statement, which shall be submitted by the Comptroller General with his report.

[The expenses for each such audit may be paid from moneys advanced therefor by the Corporation, or from any appropriation or appropriations for the General Accounting Office, and appropriations so used shall be reimbursed promptly by the Corporation as billed by the Comptroller General. All such audit expenses shall be charged to operating expenses of the Corporation.]

The expenses for each such audit shall be paid from any appropriation or appropriations for the General Accounting Office, and such part of such expenses as may be allocated to the cost of generating, transmitting, and distributing electric energy shall be reimbursed promptly by the Corporation as billed by the Comptroller General. The Comptroller General shall make special report to the President of the United States and to the Congress of any transaction or condition found by him to be in conflict with the powers or duties entrusted to the Corporation by law.

SEC. 9a. The board is hereby directed in the operation of any dam or reservoir in its possession and control to regulate the stream flow primarily for the purposes of promoting navigation and controlling floods. So far as may be consistent with such purposes, the board is authorized to provide and operate facilities for the generation of electric energy at any such dam for the use of the Corporation and for the use of the United States or any agency thereof; and the board is further authorized, whenever an opportunity is afforded, to

Material added by the 1935 amendment is underlined.
Material deleted by the 1935 amendment is enclosed in brackets.

provide and operate facilities for the generation of electric energy in order to avoid the waste of water power, to transmit and market such power as in this act provided, and thereby, so far as may be practicable, to assist in liquidating the cost or aid in the maintenance of the projects of the Authority.

SEC. 10. The board is hereby empowered and authorized to sell the surplus power not used in its operations, and for operation of locks and other works generated by it, to States, counties, municipalities, corporations, partnerships, or individuals, according to the policies hereinafter set forth; and to carry out said authority, the board is authorized to enter into contracts for such sale for a term not exceeding twenty years, and in the sale of such current by the board it shall give preference to States, counties, municipalities, and cooperative organizations of citizens or farmers, not organized or doing business for profit, but primarily for the purpose of supplying electricity to its own citizens or members: *Provided*, That all contracts made with private companies or individuals for the sale of power, which power is to be resold for a profit, shall contain a provision authorizing the board to cancel said contract upon five years' notice in writing, if the board needs said power to supply the demands of States, counties, or municipalities. In order to promote and encourage the fullest possible use of electric light and power on farms within reasonable distance of any of its transmission lines the board in its discretion shall have power to construct transmission lines to farms and small villages that are not otherwise supplied with electricity at reasonable rates, and to make such rules and regulations governing such sale and distribution of such electric power as in its judgment may be just and equitable: *Provided further*, That the board is hereby authorized and directed to make studies, experiments, and determinations to promote the wider and better use of electric power for agricultural and domestic use, or for small or local industries, and it may cooperate with State governments, or their subdivisions or agencies, with educational or research institutions, and with cooperatives or other organizations, in the application of electric power to the fuller and better balanced development of the resources of the region: *Provided further*, That the board is authorized to include in any contract for the sale of power such terms and conditions, including resale rate schedules, and to provide for such rules and regulations as in its judgment may be necessary or desirable for carrying out the purposes of this Act, and in case the purchaser shall fail to comply with any such terms and conditions, or violate any such rules and regulations, said contract may provide that it shall be voidable at the election of the board: *Provided further*, That in order to supply farms and small villages with electric power directly as contemplated by this section, the board in its discretion shall have power to acquire existing electric facilities used in servng such farms and small vllages: *And provided further*, That the terms "States", "counties", and "municipalities" as used in this Act shall be construed to include the public agencies of any of them unless the context requires a different construction.

SEC. 11. It is hereby declared to be the policy of the Government so far as practical to distribute and sell the surplus power generated at Muscle Shoals equitably among the States, counties, and municipalities within transmission distance. This policy is further declared to be that the projects herein provided for shall be considered primarily as for the benefit of the people of the section as a whole and particularly the domestic and rural consumers to whom the power can economically be made available, and accordingly that sale to and use by industry shall be a secondary purpose, to be utilized principally to secure a sufficiently high load factor and revenue returns which will permit domestic and rural use at the lowest possible rates and in such manner as to encourage increased domestic and rural use of electricity. It is further hereby declared to be the policy of the Government to utilize the Muscle Shoals properties so far as may be necessary to improve, increase, and cheapen the production of fertilizer and fertilizer ingredients by carrying out the provisions of this Act.

SEC. 12. In order to place the board upon a fair basis for making such contracts and for receiving bids for the sale of such power, it is hereby expressly

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authorized, either from appropriations made by Congress or from funds secured from the sale of such power, or from funds secured by the sale of bonds hereafter provided for, to construct, lease, purchase, or authorize the construction of transmission lines within transmission distance from the place where generated, and to interconnect with other systems. The board is also authorized to lease to any person, persons, or corporation the use of any transmission line owned by the Government and operated by the board, but no such lease shall be made that in any way interferes with the use of such transmission line by the board: *Provided*, That if any State, county, municipality, or other public or cooperative organization of citizens or farmers, not organized or doing business for profit, but primarily for the purpose of supplying electricity to its own citizens or members, or any two or more of such municipalities or organizations, shall construct or agree to construct and maintain a properly designed and built transmission line to the Government reservation upon which is located a Government generating plant, or to a main transmission line owned by the Government or leased by the board and under the control of the board, the board is hereby authorized and directed to contract with such State, county, municipality, or other organization, or two or more of them, for the sale of electricity for a term not exceeding thirty years; and in any such case the board shall give to such State, county, municipality, or other organization ample time to fully comply with any local law now in existence or hereafter enacted providing for the necessary legal authority for such State, county, municipality, or other organization to contract with the board for such power: *Provided further*, That all contracts entered into between the Corporation and any municipality or other political subdivision or cooperative organization shall provide that the electric power shall be sold and distributed to the ultimate consumer without discrimination as between consumers of the same class, and such contract shall be voidable at the election of the board if a discriminatory rate, rebate, or other special concession is made or given to any consumer or user by the municipality or other political subdivision or cooperative organization: *And provided further*, That as to any surplus power not so sold as above provided to States, counties, municipalities, or other said organizations, before the board shall sell the same to any person or corporation engaged in the distribution and resale of electricity for profit, it shall require said person or corporation to agree that any resale of such electric power by said person or corporation shall be made to the ultimate consumer of such electric power at prices that shall not exceed a schedule fixed by the board from time to time as reasonable, just, and fair; and in case of any such sale, if an amount is charged the ultimate consumer which is in excess of the price so deemed to be just, reasonable, and fair by the board, the contract for such sale between the board and such distributor of electricity shall be voidable at the election of the board: *And provided further*, That the board is hereby authorized to enter into contracts with other power systems for the mutual exchange of unused excess power upon suitable terms, for the conservation of stored water, and as an emergency or break-down relief.

SEC. 12a. In order (1) to facilitate the disposition of the surplus power of the Corporation according to the policies set forth in this Act; (2) to give effect to the priority herein accorded to States, counties, municipalities, and nonprofit organizations in the purchase of such power by enabling them to acquire facilities for the dstribution of such power; and (3) at the same time to preserve existing distribution facilities as going concerns and avoid duplication of such facilities, the board is authorized to advise and cooperate with and assist, by extending credit for a period of not exceeding five years to States, counties, and municipalities and nonprofit organizations situated within transmission distance from any dam where such power is generated by the Corporation in acquiring, improving, and operating (a) existing distribution facilities and incidental works, including generating plants; and (b) interconnecting transmission lines; or in acquiring any interest in such facilities, incidental works, and lines.

SEC. 13. Five per centum of the gross proceeds received by the board for the sale of power generated at Dam Numbered 2, or from any other hydropower plant hereafter constructed in the State of Alabama, shall be paid to the State

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of Alabama; and 5 per centum of the gross proceeds from the sale of power generated at Cove Creek Dam, hereinafter provided for, or any other dam located in the State of Tennessee, shall be paid to the State of Tennessee. Upon the completion of said Cove Creek Dam the board shall ascertain how much additional power is thereby generated at Dam Numbered 2 and at any other dam hereafter constructed by the Government of the United States on the Tennessee River, in the State of Alabama, or in the State of Tennessee, and from the gross proceeds of the sale of such additional power 2½ per centum shall be paid to the State of Alabama and 2½ per centum to the State of Tennessee. These percentages shall apply to any other dam that may hereafter be constructed and controlled and operated by the board on the Tennessee River or any of its tributaries, the main purpose of which is to control flood waters and where the development of electric power is incidental to the operation of such flood-control dam. In ascertaining the gross proceeds from the sale of such power upon which a percentage is paid to the States of Alabama and Tennessee, the board shall not take into consideration the proceeds of any power sold or delivered to the Government of the United States, or any department or agency of the Government of the United States, used in the operation of any locks on the Tennessee River or for any experimental purposes, or for the manufacture of fertilizer or any of the ingredients thereof, or for any other governmental purpose: *Provided*, That the percentages to be paid to the States of Alabama and Tennessee, as provided in this section, shall be subject to revision and change by the board, and any new percentages established by the board, when approved by the President, shall remain in effect until and unless again changed by the board with the approval of the President. No change of said percentages shall be made more often than once in five years, and no change shall be made without giving to the States of Alabama and Tennessee an opportunity to be heard.

SEC. 14. The board shall make a thorough investigation as to the present value of Dam Numbered 2, and the steam plants at nitrate plant numbered 1, and nitrate plant numbered 2, and as to the cost of Cove Creek Dam, for the purpose of ascertaining how much of the value or the cost of said properties shall be allocated and charged up to (1) flood control, (2) navigation, (3) fertilizer, (4) national defense, and (5) the development of power. The findings thus made by the board, when approved by the President of the United States, shall be final, and such findings shall thereafter be used in all allocations of value for the purpose of keeping the book value of said properties. In like manner, the cost and book value of any dams, steam plants, or other similar improvements hereafter constructed and turned over to said board for the purpose of control and management shall be ascertained and allocated.

The board shall, on or before January 1, 1937, file with Congress a statement of its allocation of the value of such properties turned over to said board and which have been completed prior to the end of the preceding fiscal year, and shall thereafter in its annual report to Congress file a statement of its allocation of the value of such properties as have been completed during the preceding fiscal year.

For the purpose of accumulating data useful to the Congress in the formulation of legislative policy in matters relating to the generation, transmission, and distribution of electric energy and the production of chemicals necessary to national defense and useful in agriculture, and to the Federal Power Commission and other Federal and State agencies, and to the public, the board shall keep complete accounts of its costs of generation, transmission, and distribution of electric energy and shall keep a complete account of the total cost of generating and transmission facilities constructed or otherwise acquired by the Corporation, and of producing such chemicals, and a description of the major components of such costs according to such uniform system of accounting for public utilities as the Federal Power Commission has, and if it have none, then it is hereby empowered and directed to prescribe such uniform system of accounting, together with records of such other physical data and operating statistics of the Authority as may be helpful in determining the

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actual cost and value of services, and the practices, methods, facilities, equipment, appliances, and standards and sizes, types, location, and geographical and economic integration of plants and systems best suited to promote the public interest, efficiency, and the wider and more economical use of electric energy. Such data shall be reported to the Congress by the board from time to time with appropriate analyses and recommendations, and, so far as practicable, shall be made available to the Federal Power Commission and other Federal and State agencies which may be concerned with the administration of legislation relating to the generation, transmission, or distribution of electric energy and chemicals useful to agriculture. It is hereby declared to be the policy of this Act that, in order, as soon as practicable, to make the power projects self-supporting and self-liquidating, the surplus power shall be sold at rates which, in the opinion of the board, when applied to the normal capacity of the Authority's power facilities, will produce gross revenues in excess of the cost of production of said power and in addition to the statement of the cost of power at each power station as required by section 9 (a) of the "Tennessee Valley Act of 1933," the board shall file with each annual report a statement of the total cost of all power generated by it at all power stations during each year, the average cost of such power per kilowatt-hour, the rates at which sold, and to whom sold, and copies of all contracts for the sale of power.

SEC. 15. In the construction of any future dam, steam plant, or other facility, to be used in whole or in part for the generation or transmission of electric power the board is hereby authorized and empowered to issue on the credit of the United States and to sell serial bonds not exceeding $50,000,000 in amount, having a maturity not more than fifty years from the date of the issue thereof, and bearing interest not exceeding 3½ per centum per annum. Said bonds shall be issued and sold in amounts and prices approved by the Secretary of the Treasury, but all such bonds as may be so issued and sold shall have equal rank. None of said bonds shall be sold below par, and no fee, commission, or compensation whatever shall be paid to any person, firm, or corporation for handling, negotiating the sale, or selling the said bonds. All of such bonds so issued and sold shall have all the rights and privileges accorded by law to Panama Canal bonds, authorized by section 8 of the act of June 28, 1902, chapter 1302, as amended by the act of December 21, 1905 (ch. 3, sec. 1, 34 Stat. 5), as now compiled in section 743 of title 31 of the United States Code. All funds derived from the sale of such bonds shall be paid over to the Corporation.

SEC. 15a. With the approval of the Secretary of the Treasury, the Corporation is authorized to issue bonds not to exceed in the aggregate $50,000,000 outstanding at any one time, which bonds may be sold by the Corporation to obtain funds to carry out the provisions of section 12a of this Act. Such bonds shall be in such forms and denominations, shall mature within such periods not more than fifty years from the date of their issue, may be redeemable at the option of the Corporation before maturity in such manner as may be stipulated therein, shall bear such rates of interest not exceeding 3½ per centum per annum, shall be subject to such terms and conditions, shall be issued in such manner and amount, and sold at such prices, as may be prescribed by the Corporation with the approval of the Secretary of the Treasury: *Provided*, That such bonds shall not be sold at such prices or on such terms as to afford an investment yield to the holders in excess of 3½ per centum per annum. Such bonds shall be fully and unconditionally guaranteed both as to interest and principal by the United States, and such guaranty shall be expressed on the face thereof, and such bonds shall be lawful investments, and may be accepted as security, for all fiduciary, trust, and public funds, the investment or deposit of which shall be under the authority or control of the United States or any officer or officers thereof. In the event that the Corporation should not pay

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upon demand, when due, the principal of, or interest on, such bonds, the Secretary of the Treasury shall pay to the holder the amount thereof, which is hereby authorized to be appropriated out of any moneys in the Treasury not otherwise appropriated. and thereupon to the extent of the amount so paid the Secretary of the Treasury shall succeed to all the rights of the holders of such bonds. The Secretary of the Treasury, in his discretion, is authorized to purchase any bonds issued hereunder, and for such purpose the Secretary of the Treasury is authorized to use as a public-debt transaction the proceeds from the sale of any securities hereafter issued under the Second Liberty Bond Act, as amended, and the purposes for which securities may be issued under such Act, as amended, are extended to include any purchases of the Corporation's bonds hereunder. The Secretary of the Treasury may, at any time, sell any of the bonds of the Corporation acquired by him under this section. All redemptions, purchases, and sales by the Secretary of the Treasury of the bonds of the Corporation shall be treated as public-debt transactions of the United States. With the approval of the Secretary of the Treasury, the Corporation shall have power to purchase such bonds in the open market at any time and at any price. No bonds shall be issued hereunder to provide funds or bonds necessary for the performance of any proposed contract negotiated by the Corporation under the authority of section 12a of this Act until the proposed contract shall have been submitted to and approved by the Federal Power Commission. When any such proposed contract shall have been submitted to the said Commission, the matter shall be given precedence and shall be in every way expedited and the Commission's determination of the matter shall be final. The authority of the Corporation to issue bonds hereunder shall expire at the end of five years from the date when this section as amended herein becomes law, except that such bonds may be issued at any time after the expiration of said period to provide bonds or funds necessary for the performance of any contract entered into by the Corporation, prior to the expiration of said period, under the authority of section 12a of this Act.

SEC. 16. The board, whenever the President deems it advisable, is hereby empowered and directed to complete Dam Numbered 2 at Muscle Shoals, Alabama, and the steam plant at nitrate plant numbered 2, in the vicinity of Muscle Shoals, by installing in Dam Numbered 2 the additional power units according to the plans and specifications of said dam, and the additional power unit in the steam plant at nitrate plant numbered 2.

SEC. 17. The Secretary of War, or the Secretary of the Interior, is hereby authorized to construct, either directly or by contract to the lowest responsible bidder, after due advertisement, a dam in and across Clinch River in the State of Tennessee, which has by long custom become known and designated as the Cove Creek Dam, together with a transmission line from Muscle Shoals, according to the latest and most approved designs, including powerhouse and hydroelectric installations and equipment for the generation of power, in order that the waters of said Clinch River may be impounded and stored above said dam for the purpose of increasing and regulating the flow of the Clinch River and the Tennessee River below, so that the maximum amount of primary power may be developed at Dam Numbered 2 and at any and all other dams below the said Cove Creek Dam: *Provided, however*, That the President is hereby authorized by appropriate order[1] to direct the employment by the Secretary of War, or by the Secretary of the Interior, of such engineer or engineers as he may designate, to perform such duties and obligations as he may deem proper, either in the drawing of plans and specifications for said dam, or to perform any other work in the building or construction of the same. The President may, by such order, place the control of the construction of said dam in the hands of such engineer or engineers taken from private life as he may desire:

Material added by the 1935 amendment is underlined.

[1] Executive Order Numbered 6162 (see p. 825) placed the construction in the charge of Arthur E. Morgan.

And provided further, That the President is hereby expressly authorized without regard to the restriction or limitation of any other statute, to select attorneys and assistants for the purpose of making any investigation as he may deem proper to ascertain whether, in the control and management of Dam Numbered 2, or any other dam or property owned by the Government in the Tennessee River Basin, or in the authorization of any improvement therein, there has been any undue or unfair advantage given to private persons, partnerships, or corporations by any officials or employees of the Government, or whether in any such matters the Government has been injured or unjustly deprived of any of its rights.

SEC. 18. In order to enable and empower the Secretary of War, the Secretary of the Interior, or the board to carry out the authority hereby conferred in the most economical and efficient manner, he or it is hereby authorized and empowered in the exercise of the powers of national defense in aid of navigation, and in the control of the flood waters of the Tennessee and Mississippi Rivers, constituting channels of interstate commerce, to exercise the right of eminent domain for all purposes of this Act and to condemn all lands, easements, rights-of-way, and other area necessary in order to obtain a site for said Cove Creek Dam, and the flowage rights for the reservoir of water above said dam, and to negitiate and conclude contracts with States, counties, municipalities, and all State agencies, and with railroads, railroad corporations, common carriers, and all public-utility commissions, and any other person, firm, or corporation, for the relocation of railroad tracts, highways, highway bridges, mills, ferries, electric-light plants, and any and all other properties, enterprises, and projects whose removal may be necessary in order to carry out the provisions of this Act. When said Cove Creek Dam, transmission line, and powerhouse shall have been completed, the possession, use, and control thereof shall be intrusted to the corporation for use and operation in connection with the general Tennessee Valley project and to promote flood control and navigation in the Tennessee River.

SEC. 19. The Corporation, as an instrumentality and agency of the Government of the United States for the purpose of executing its constitutional powers, shall have access to the Patent Office of the United States for the purpose of studying, ascertaining, and copying all methods, formulae, and scientific information (not including access to pending applications for patents) necessary to enable the Corporation to use and employ the most efficacious and economical process for the production of fixed nitrogen, or any essential ingredient of fertilizer, or any method of improving and cheapening the production of hydroelectric power, and any owner of a patent whose patent rights may have been thus in any way copied, used, infringed, or employed by the exercise of this authority by the Corporation shall have as the exclusive remedy a cause of action against the Corporation, to be instituted and prosecuted on the equity side of the appropriate district court of the United States for the recovery of reasonable compensation for such infringement. The Commissioner of Patents shall furnish to the Corporation, at its request and without payment of fees, copies of documents on file in his office: *Provided,* That the benefits of this section shall not apply to any art, machine, method of manufacture, or composition of matter discovered or invented by such employee during the time of his employment or service with the Corporation or with the Government of the United States.

SEC. 20. The Government of the United States hereby reserves the right, in case of war or national emergency declared by Congress, to take possession of all or any part of the property described or referred to in this Act for the purpose of manufacturing explosives or for other war purposes; but, if this right is exercised by the Government, it shall pay the reasonable and fair damages that may be suffered by any party whose contract for the purchase of electric power or fixed nitrogen or fertilizer ingredients is hereby violated, after the amount of the damages has been fixed by the United States Court of Claims in proceedings instituted and conducted for that purpose under rules prescribed by the court.

SEC. 21. (a) All general penal statutes relating to the larceny, embezzlement, conversion, or to the improper handling, retention, use, or disposal of public moneys or property of the United States, shall apply to the moneys and property of the Corporation and to moneys and properties of the United States intrusted to the Corporation.

(b) Any person who, with intent to defraud the Corporation, or to deceive any director, officer, or employee of the Corporation or any officer or employee

of the United States (1) makes any false entry in any book of the Corporation, or (2) makes any false report or statement for the Corporation, shall, upon conviction thereof, be fined not more than $10,000 or imprisoned not more than five years, or both.

(c) Any person who shall receive any compensation, rebate, or reward, or shall enter into any conspiracy, collusion, or agreement, express or implied, with intent to defraud the Corporation or wrongfully and unlawfully to defeat its purposes, shall, on conviction thereof, be fined not more than $5,000 or imprisoned not more than five years, or both.

SEC. 22. To aid further the proper use, conservation, and development of the natural resources of the Tennessee River drainage basin and of such adjoining territory as may be related to or materially affected by the development consequent to this act, and to provide for the general welfare of the citizens of said areas, the President is hereby authorized, by such means or methods as he may deem proper within the limits of appropriations made therefor by Congress, to make such surveys of and general plans for said Tennessee basin and adjoining territory as may be useful to the Congress and to the several States in guiding and controlling the extent, sequence, and nature of development that may be equitably and economically advanced through the expenditure of public funds, or through the guidance or control of public authority, all for the general purpose of fostering an orderly and proper physical, economic, and social development of said areas; and the President is further authorized in making said surveys and plans to cooperate with the States affected thereby, or subdivisions or agencies of such States, or with cooperative or other organizations, and to make such studies, experiments, or demonstrations as may be necessary and suitable to that end.

SEC. 23. The President shall, from time to time, as the work provided for in the preceding section progresses, recommend to Congress such legislation as he deems proper to carry out the general purposes stated in said section, and for the especial purpose of bringing about in said Tennessee drainage basin and adjoining territory in conformity with said general purposes (1) the maximum amount of flood control; (2) the maximum development of said Tennessee River for navigation purposes; (3) the maximum generation of electric power consistent with flood control and navigation; (4) the proper use of marginal lands; (5) the proper method of reforestation of all lands in said drainage basin suitable for reforestation; and (6) the economic and social well-being of the people living in said river basin.

SEC. 24. For the purpose of securing any rights of flowage, or obtaining title to or possession of any property, real or personal, that may be necessary or may become necessary, in the carrying out of any of the provisions of this Act, the President of the United States for a period of three years from the date of the enactment of this Act, is hereby authorized to acquire title in the name of the United States to such rights or such property, and to provide for the payment for same by directing the board to contract to deliver power generated at any of the plants now owned or hereafter owned or constructed by the Government or by said Corporation, such future delivery of power to continue for a period not exceeding thirty years. Likewise, for one year after the enactment of this Act, the President is further authorized to sell or lease any parcel or part of any vacant real estate now owned by the Government in said Tennessee River Basin to persons, firms, or corporations who shall contract to erect thereon factories or manufacturing establishments, and who shall contract to purchase of said Corporation electric power for the operation of any such factory or manufacturing establishment. No contract shall be made by the President for the sale of any of such real estate as may be necessary for present or future use on the part of the Government for any of the purposes of this Act. Any such contract made by the President of the United States shall be carried out by the board: *Provided*, That no such contract shall be made that will in any way abridge or take away the preference right to purchase power given in this Act to States, counties, municipalities, or farm organizations: *Provided further*, That no lease shall be for a term to exceed fifty years: *Provided further*, That any sale shall be on condition that said land shall be used for industrial purposes only.

SEC. 25. The Corporation may cause proceedings to be instituted for the acquisition by condemnation of any lands, easements, or rights-of-way which, in the opinion of the Corporation, are necessary to carry out the provisions of this Act. The proceedings shall be instituted in the United States district court for

the district in which the land, easement, right-of-way, or other interest, or any part thereof, is located, and such court shall have full jurisdiction to divest the complete title to the property sought to be acquired out of all persons or claimants and vest the same in the United States in fee simple, and to enter a decree quieting the title thereto in the United States of America.

Upon the filing of a petition for condemnation and for the purpose of ascertaining the value of the property to be acquired, and assessing the compensation to be paid, the court shall appoint three commissioners who shall be disinterested persons and who shall take and subscribe an oath that they do not own any lands, or interest or easement in any lands, which it may be desirable for the United States to acquire in the furtherance of said project, and such commissioners shall not be selected from the locality wherein the land sought to be condemned lies. Such commissioners shall receive a per diem of not to exceed $15 for their services, together with an additional amount of $5 per day for subsistence for time actually spent in performing their duties as commissioners.

It shall be the duty of such commissioners to examine into the value of the lands sought to be condemned, to conduct hearings and receive evidence, and generally to take such appropriate steps as may be proper for the determination of the value of the said lands sought to be condemned, and for such purpose the commissioners are authorized to administer oaths and subpoena witnesses, which said witnesses shall receive the same fees as are provided for witnesses in the Federal courts. The said commissioners shall thereupon file a report setting forth their conclusions as to the value of the said property sought to be condemned, making a separate award and valuation in the premises with respect to each separate parcel involved. Upon the filing of such award in court the clerk of said court shall give notice of the filing of such award to the parties to said proceeding, in manner and form as directed by the judge of said court.

Either or both parties may file exceptions to the award of said commissioners within twenty days from the date of the filing of said award in court. Exceptions filed to such award shall be heard before three Federal district judges unless the parties, in writing, in person, or by their attorneys, stipulate that the exceptions may be heard before a lesser number of judges. On such hearing such judges shall pass de novo upon the proceedings had before the commissioners, may view the property, and may take additional evidence. Upon such hearings the said judges shall file their own award, fixing therein the value of the property sought to be condemned, regardless of the award previously made by the said commissioners.

At any time within thirty days from the filing of the decision of the district judges upon the hearing on exceptions to the award made by the commissioners, either party may appeal from such decision of the said judges to the circuit court of appeals, and the said circuit court of appeals shall upon the hearing on said appeal dispose of the same upon the record, without regard to the awards or findings theretofore made by the commissioners or the district judges, and such circuit court of appeals shall thereupon fix the value of the said property sought to be condemned.

Upon acceptance of an award by the owner of any property herein provided to be appropriated, and the payment of the money awarded or upon the failure of either party to file exceptions to the award of the commissioners within the time specified, or upon the award of the commissioners, and the payment of the money by the United States pursuant thereto, or the payment of the money awarded into the registry of the court by the Corporation, the title to said property and the right to the possession thereof shall pass to the United States, and the United States shall be entitled to a writ in the same proceeding to dispossess the former owner of said property, and all lessees, agents, and attorneys of such former owner, and to put the United States, by its corporate creature and agent, the Corporation, into possession of said property.

In the event of any property owned in whole or in part by minors, or insane persons, or incompetent persons, or estates of deceased persons, then the legal representatives of such minors, insane persons, incompetent persons, or estates shall have power, by and with the consent and approval of the trial judge in whose court said matter is for determination, to consent to or reject the awards of the commissioners herein provided for, and in the event that there be no legal representatives, or that the legal representatives for such minors, insane persons, or incompetent persons shall fail or decline to act, then such trial

judge may, upon motion, appoint a guardian ad litem to act for such minors, insane persons, or incompetent persons, and such guardian ad litem shall act to the full extent and to the same purpose and effect as his ward could act, if competent, and such guardian ad litem shall be deemed to have full power and authority to respond, to conduct, or to maintain any proceeding herein provided for affecting his said ward.

[SEC. 26. The net proceeds derived by the board from the sale of power and any of the products manufactured by the Corporation, after deducting the cost of operation, maintenance, depreciation, amortization, and an amount deemed by the board as necessary to withhold as operating capital, or devoted by the board to new construction, shall be paid into the Treasury of the United States at the end of each calendar year.]

SEC. 26. Commencing July 1, 1936, the proceeds for each fiscal year derived by the board from the sale of power or any other products manufactured by the Corporation, and from any other activities of the Corporation including the disposition of any real or personal property, shall be paid into the Treasury of the United States at the end of each calendar year, save and except such part of such proceeds as in the opinion of the board shall be necessary for the Corporation in the operation of dams and reservoirs, in conducting its business in generating, transmitting, and distributing electric energy and in manufacturing, selling, and distributing fertilizer and fertilizer ingredients. A continuing fund of $1,000,000 is also excepted from the requirements of this section and may be withheld by the board to defray emergency expenses and to insure continuous operation: *Provided*, That nothing in this section shall be construed to prevent the use by the board, after June 30, 1936, of proceeds accruing prior to July 1, 1936, for the payment of obligations lawfully incurred prior to such latter date.

SEC. 26a. The unified development and regulation of the Tennessee River system requires that no dam, appurtenant works, or other obstruction affecting navigation, flood control, or public lands or reservations shall be constructed, and thereafter operated or maintained across, along, or in the said river or any of its tributaries until plans for such construction, operation, and maintenance shall have been submitted to and approved by the board; and the construction, commencement of construction, operation, or maintenance of such structures without such approval is hereby prohibited. When such plans shall have been approved, deviation therefrom either before or after completion of such structures is prohibited unless the modification of such plans has previously been submitted to and approved by the board.

In the event the board shall, within sixty days after their formal submission to the board, fail to approve any plans or modifications, as the case may be, for construction, operation, or maintenance of any such structures on the Little Tenessee River, the above requirements shall be deemed satisfied, if upon application to the Secretary of War, with due notice to the Corporation, and hearing thereon, such plans or modifications are approved by the said Secretary of War as reasonably adequate and effective for the unified development and regulation of the Tennessee River system.

Such construction, commencement of construction, operation, or maintenance of any structures or parts thereof in violation of the provisions of this section may be prevented, and the removal or discontinuation thereof required by the injunction or order of any district court exercising jurisdiction in any district in which such structures or parts thereof may be situated, and the Corporation is hereby authorized to bring appropriate proceedings to this end.

Material added by the 1935 amendment is underlined.
Material deleted by the 1935 amendment is enclosed in brackets

The requirements of this section shall not be construed to be a substitute for the requirements of any other law of the United States or of any State now in effect or hereafter enacted, but shall be in addition thereto, so that any approval, license, permit, or other sanction now or hereafter required by the provisions of any such law for the construction, operation, or maintenance of any structures whatever, except such as may be constructed, operated, or maintained by the Corporation, shall be required, notwithstanding the provisions of this section.

SEC. 27. All appropriations necessary to carry out the provisions of this Act are hereby authorized.

SEC. 28. That all Acts or parts of Acts in conflict herewith are hereby repealed, so far as they affect the operations contemplated by this Act.

SEC. 29. The right to alter, amend, or repeal this Act is hereby expressly declared and reserved, but no such amendment or repeal shall operate to impair the obligation of any contract made by said Corporation under any power conferred by this Act.

SEC. 30.[2] The sections of this Act are hereby declared to be separable, and in the event any one or more sections of this Act be held to be unconstitutional, the same shall not affect the validity of other sections of this Act.

SEC. 31. This Act shall be liberally construed to carry out the purposes of Congress to provide for the disposition of and make needful rules and regulations respecting Government properties entrusted to the Authority, provide for the national defense, improve navigation, control destructive floods, and promote interstate commerce and the general welfare, but no real estate shall be held except what is necessary in the opinion of the board to carry out plans and projects actually decided upon requiring the use of such land: *Provided*, That any land purchased by the Authority and not necessary to carry out plans and projects actually decided upon shall be sold by the Authority as agent of the United States, after due advertisement, at public auction to the highest bidder, or at private sale as provided in section 4 (k) of this Act.

EXECUTIVE ORDER

CONSERVATION AND DEVELOPMENT OF THE NATURAL RESOURCES OF THE TENNESSEE RIVER DRAINAGE BASIN

In accordance with the provision of section 22 and section 23 of the Tennessee Valley Authority Act of 1933, the President hereby authorizes and directs the Board of Directors of the Tennessee Valley Authority to make such surveys, general plans, studies, experiments, and demonstrations as may be necessary and suitable to aid the proper use, conservation, and development of the natural resources of the Tennessee River drainage basin, and of such adjoining territory as may be related to or materially affected by the development consequent to this act, and to promote the general welfare of the citizens of said area; within the limits of appropriations made therefore by Congress.

FRANKLIN D. ROOSEVELT.

THE WHITE HOUSE,
June 8, 1933.

[No. 6161]

Material added by the 1935 amendment is underlined.

[2] **Section 15 of the amendatory Act provides as follows:**

"That the sections of this Act are hereby declared to be separable, and in the event of any one or more sections of this Act, or parts thereof, be held to be unconstitutional such holding shall not affect the validity of other sections or parts of this Act."

EXECUTIVE ORDER

CONSTRUCTION OF COVE CREEK DAM ON CLINCH RIVER

In accordance with the provisions of sections 17 and 18 of the Tennessee Valley Authority Act of 1933, I do hereby place the construction of the Cove Creek Dam on Clinch River in the hands of Arthur E. Morgan without additional compensation to said Arthur E. Morgan, and under his direction of such engineers as may be necessary for that purpose, with the understanding that the work shall be done by and through the Tennessee Valley Authority.

FRANKLIN D. ROOSEVELT.

THE WHITE HOUSE,
June 8, 1933.

[No. 6162]

EXECUTIVE ORDER

RATES OF COMPENSATION OF GOVERNMENT EMPLOYEES IN EMERGENCY AGENCIES NOT SUBJECT TO THE CLASSIFICATION ACT, AND ACTS AMENDATORY THEREOF

By virtue of the authority vested in me as President of the United States, and under authority of section 1753 of the Revised Statutes, and in order to secure greater uniformity in the rates of compensation of employees engaged upon the same or similar classes of work, it is hereby ordered that the respective heads of

National Industrial Recovery Administration,
Agricultural Adjustment Administration,
Tennessee Valley Authority,
Public Works Administration,
Emergency Conservation Work,
Reconstruction Finance Corporation,
Farm Credit Administration,
Federal Emergency Relief Administration,
Home Owners' Loan Corporation,
Federal Coordinator of Transportation,

which are authorized by law to employ personal services and to fix rates of compensation therefor without regard to the Classification Act of 1923, as amended, shall, unless otherwise specifically authorized by me, fix such rates for officers and employees now in the service, as well as for those hereafter appointed, at the amounts prescribed in the following salary-standardization schedule, in accordance with the duties and responsibilities of the positions occupied by them:

Salary schedule, emergency agencies

Grade	Salary	Corresponding services and grades under amended Classification Act	Grade	Salary	Corresponding services and grades under amended Classification Act
1	$840	CU-1 ($600).	10	$2,900	CAF-8.
2	$1,080	CU-2.	11	$3,200	P-3. CAF-9.
3	$1,260	CAF-1. SP-2.	12	$3,600	CAF-10 ($3,500).
4	$1,440	CAF-2. SP-3.	13	$4,000	P-4 CAF-11 ($3,800).
5	$1,620	CAF-3. SP-4.	14	$4,500	P-5 CAF-12 ($4,600).
6	$1,800	CAF-4. SP-5.	15	$5,200	P-6 CAF-13 ($5,600).
7	$2,000	P-1; CAF-5. CU-8; SP-6.	16	$6,000	P-7 CAF-14 ($6,500).
8	$2,300	CAF-6. CU-9. SP-7.	17	$6,800	
			18	$8,000	P-8. CAF-15.
9	$2,600	P-2; CAF-7. CU-10; SP-8.	19	Over $8,000	P-9 CAF-16 (over $9,000).

In all cases not subject to the provisions of section 2, title II, of the act of March 20, 1933 (Public, No. 2, 73d Cong.), the rates payable under this schedule shall be decreased by the amount, if any, by which such rate would be reduced

pursuant to the provisions of said section 2 if such section were applicable. All appointments hereafter shall stipulate that the rate to be paid thereunder shall be the appropriate specific rate of the schedule less the amount, if any, by which such rate would be reduced pursuant to the provisions of said section 2 if such section were applicable.

In making the initial adjustment of salaries under this order, where it is necessary to reduce the present rate of an employee to conform to the rate scheduled for the grade to which his position is allocated under the salary schedule herein approved, the head of the department, establishment, or agency concerned may fix such basic rate at an amount not more than 20 percent in excess of such scheduled rate (*a*) where the employee is occupying a position allocated to grade 13 or higher in the said salary schedule, or (*b*) where the employee was reinstated or transferred under the civil-service rules to his present position from a regular Government establishment (or could have been so reinstated or transferred), or where the employee was selected from the reemployment list of the Civil Service Commission, provided such 20 percent excess, or part thereof, shall be applied only insofar as is necessary to increase the basic salary rate of the employee so reinstated, transferred, or reemployed, as fixed under the said salary schedule, to an amount not in excess of the basic salary rate last received by him in the department or establishment in which he was previously employed.

Such adjustments in existing rates of compensation as may be necessary to comply with the provisions of this order shall be made effective as soon hereafter as may be practicable.

FRANKLIN D. ROOSEVELT.

THE WHITE HOUSE,
November 18, 1933.

[No. 6440]

EXECUTIVE ORDER

RATES OF COMPENSATION OF GOVERNMENT EMPLOYEES IN EMERGENCY AGENCIES, ETC., NOT SUBJECT TO THE CLASSIFICATION ACT AS AMENDED

By virtue of and pursuant to the authority vested in me as President of the United States, it is hereby ordered that the heads of existing emergency agencies and of those hereafter created and (except the heads of executive departments and independent establishments) the heads of all other agencies operated in whole or in part from emergency funds, the compensation of the employees of which may be fixed without regard to the Classification Act of 1923, as amended, shall, unless otherwise specifically authorized by me, classify the positions of the employees of their respective agencies now in the service or hereafter appointed in accordance with the following salary schedule and adjust and fix the rates of compensation therefor at amounts not in excess of those prescribed therein for the corresponding grades:

Salary schedule

Grade	Salary	Corresponding services and grades under amended Classification Act	Grade	Salary	Corresponding services and grades under amended Classification Act
1	$840	CU-1 ($600).	10	$2,900	CAF-8.
2	$1,080	CU-2.	11	$3,200	P-3. CAF-9.
3	$1,260	CAF-1. SP-2.	12	$3,600	CAF-10 ($3,500).
4	$1,440	CAF-2. SP-3.	13	$4,000	P-4 CAF-11 ($3,800).
5	$1,620	CAF-3. SP-4.	14	$4,500	P-5 CAF-12 ($4,600).
6	$1,800	CAF-4. SP-5.	15	$5,200	P-6 CAF-13 ($5,600).
7	$2,000	P-1; CAF-5. CU-8; SP-6.	16	$6,000	P-7 CAF-14 ($6,500).
8	$2,300	CAF-6. CU-9. SP-7.	17	$6,800	P-8. CAF-15.
	$2,600	P-2; CAF-7. CU-10; SP-8.	18	$8,000	
			19	Over $8,000	P-9 CAF-16 (over $9,000).

The positions of employees in executive departments and independent establishments who are paid from emergency funds shall be classified under the provisions of the Classification Act of 1923, as amended: *Provided,* That the heads of such departments and establishments may elect to fix such rates of compensation either under the said Classification Act or in accordance with the salary schedule prescribed in this order.

All classifications made pursuant to this order, except those made in accordance with the provisions of the Classification Act of 1923, as amended, shall be subject to review and revision by the Civil Service Commission upon the request of the executive council. In the event any classification made by the head of a department or an agency pursuant to this order is revised by the Civil Service Commission, the revised classification shall be final and effective beginning the first day of the first month after such revision is reported by the executive council to the head of the department or agency concerned.

The following are hereby excepted from the provisions of this order:

(1) All employees of the Tennessee Valley Authority and its affiliates.

(2) All employees of the Office of the Federal Coordinator of Transportation.

(3) All common, unskilled, skilled or semiskilled laborers, skilled tradesmen such as machinists, plumbers, steamfitters, carpenters, painters, etc., and the foremen of single groups of such employees, and seamen and ship's officers, whose rates of compensation, under existing laws and practice, are fixed by labor wage boards, or at local wage rates as determined under authority of law by the heads of the agencies concerned.

(4) All enrollees of Emergency Conservation Work camps.

(5) All employees of the Federal Civil Works Administration and the Federal Emergency Relief Administration except the clerical and administrative personnel employed in the central offices of the said Administrations in the District of Columbia.

(6) All other persons engaged in employment similar to that of the classes of employees described in (3), (4), and (5), supra, or employed under similar circumstances.

(7) All employees engaged upon the educational program of the Emergency Conservation Work.

(8) All employees of the Regional Agricultural Credit Corporation.

In all cases not subject to the provisions of section 2, title II, of the act of March 20, 1933 (ch. 3, 48 Stat. 8, 12), as amended by section 22, title II, of the act of March 28, 1934 (Public, No. 131, 73d Cong.), the rates payable to employees subject to the provisions of this order shall be the rates fixed within the respective grades decreased by the amount, if any, by which such rate would be reduced pursuant to the provisions of said section 2 if such section were applicable. Every appointment hereafter made shall stipulate that the rate to be paid thereunder shall be the appropriate grade rate specified therein less the amount, if any, by which such rate would be reduced pursuant to the provisions of said section 2 if such section were applicable.

All adjustments in rates of compensation made pursuant to this order shall be reported to the executive council and shall become effective beginning the first day of the first month subsequent to that in which such report is made.

The term "adjustments" as used herein shall include increases as well as decreases in rates of compensation, and the term "employees" shall include officers.

This order supersedes Executive Orders Numbers 6440, 6553, and 6622, dated November 18, 1933, January 10, 1934, and March 1, 1934, respectively.

FRANKLIN D. ROOSEVELT.

THE WHITE HOUSE,
June 21, 1934.

[6746]

Appendix K

PLANS AND SPECIFICATIONS FOR THE NORRIS DAM

Issued separately by the Tennessee Valley Authority as Technical Monograph No. 34, 1938. Price $2.50.

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